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Public Health Problems in Impounding Water in the Tennessee Valley

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A Contribution

THE Tennessee Valley Authority is a federal regional agency which is building a series of multiple-purpose high-head dams and storage reservoirs for navigation, flood control and the production of electricity. These reservoirs are located on the Tennessee River and its tributaries in Alabama, Georgia, Kentucky, Mississippi, North Carolina and Tennessee. A program of regional development, planning and conservation is being carried out in conjunction with the construction of dams in order to achieve the maximum benefits for the region.

Many public health problems are created or aggravated by damming streams to form reservoirs. These problems vary as much as locations and types of reservoirs vary throughout the country. It is difficult to predict all conditions which may develop without a thorough study of each case. Previous experience at other projects indicated the necessity for a thorough investigation of all related activities (although this rounded-out program is a pioneer undertaking). It was known that water supplies, sewerage systems and recreational facilities are often affected and general stream conditions may be altered. Population redistribution, as a result of these activities, has public health implications. Malaria is associated with standing water in which the malaria mosquito may breed. Occupational health hazards often exist in connection with dam construction.

These facts were known, but the corrective measures to apply and the procedures to follow were not as clearly defined or as simply executed. The efforts of many groups within the Authority, as well as outside agencies, were required in analyzing the numerous factors associated with solving the new public health problems that have been created by TVA developments.

The Authority's Health and Safety Department was organized with the two-fold purpose of caring for the health, sanitation and safety of its employees and of serving as a contact with federal, state and local health organizations in activities in the Tennessee Valley. As is true of all TVA departments, it does not duplicate efforts of agencies already organized, but rather it supplements their efforts and aids in solving new problems created by TVA activities. Such contacts have been stimulating to the existing agencies. Active co-operation is maintained and these groups are kept informed of developments.

General Stream Conditions

A stream is closely associated with the economy of its drainage area. The general conditions of the stream are likewise important for maximum utilization of this important natural resource. The Authority has followed this conception in its program of regional development in the Tennessee Valley, and is conducting studies to determine not only the existing stream conditions, but also the changes taking place as the rivers are dammed and a series of artificial lakes is created. Many of these factors have health implications, while some are more important from other angles.

Many physical changes are taking place due to the nine river dams on the Tennessee River between its mouth and Knoxville, Tenn. Instead of a series of rapids and comparatively shallow pools for the 650 mi. of river with a total fall of about 500 ft., there will be nine comparatively constant level pools varying in depth from 75 ft. at the dams to a minimum of 9 ft. at the upper ends of the lakes. Pool elevations will be subject to fluctuations of about 10 ft. for flood control, malaria control and maximum power generation. This change from a shallow stream will greatly increase the surface area of the water course and will also increase the time of flow to at least six to ten times the previous time of flow in the main channel.

An example of such changes may be shown by comparing Wheeler Reservoir with the river in the same section before impoundage. The original river area was 17,600 acres, while the reservoir now covers about 70,000 acres and has an average width of 1.42 mi. The controlling depth in this section of the river was 3 ft. prior to 1933 while now the controlling depth is 11 ft. The depth of water at Wheeler Dam is about 76 ft. in the channel.

Estimates of increases in the time of flow vary over a wide range. Measurements of the time of flow in Fort Loudoun Reservoir during average summer flows before impoundage indicated that the river velocity was 1.3 mph., or about 1.5 days through the reservoir. Estimates since impoundage indicate that the time of flow is from 20 to 30 days for equivalent discharges. Similar values are estimated for Pickwick Reservoir. These were the only two reservoirs on which float studies were made to determine actual times of flow prior to filling the reservoirs. Velocities in reservoirs are too low to be measured by floats, and velocities calculated from cross-sections will not show the time of flow of pollution. Studies are under way at present to follow pollution through reservoirs by chemical analyses, but peaks are difficult to follow because of dispersion.

The main tributaries of the Tennessee River are being provided with storage dams which vary in size and

amount of reservoir storage. All of the dams in the Valley are being operated as a unit so that discharges and power loads can be shifted from one dam to another, depending on all conditions involved. The general scheme of operation of the tributary storage reservoirs will be to impound most of the stream flow from January until early or mid-summer, and after that to release water at a rapid rate until late fall or early winter. This will decrease the high winter flow and increase the usual low stream flow of summer and fall.

Seasonal discharge estimates and duration curves are not available at this time. When the entire TVA system is in operation, however, the tendency will be to have seasonal fluctuations on the controlled tributaries and a more even flow on the main Tennessee River where the discharge will tend to approximate the mean flow. Uncontrolled floods and low water will cease to be problems.

The Tennessee Valley, as a whole, contains relatively clean streams when compared with other regions in the United States. There are, however, a few sections of streams such as on the Pigeon, French Broad, Holston, Tuckasegee and Emory rivers and the Tennessee River below Knoxville and Chattanooga that are so seriously polluted that they are not only objectionable from the esthetic point of view, but are affecting water supplies and recreational activities. These existed prior to the changes brought about by the Authority, and many of these sections are not directly related to the new lakes. Nevertheless, all stream pollution is being investigated and should be remedied or at least alleviated before the broad program of regional development is complete.

Two reports have been prepared by the Authority's Health and Safety Department entitled, "Studies of the Pollution of the Tennessee River System." Part I (1) was published in 1941 and presents the results of studies on the main river. Part II (2) will be published in 1945 and will discuss the types and sources of pollution in the Valley and their effect on the system. These should serve as a starting point for corrective measures. Tennessee has taken the lead and a board has been appointed by the governor to investigate stream pollution in that state and report on stream sanitation to the 1945 legislature. A cooperative program is also being developed with North Carolina.

In general, the sanitary quality of the lake waters has been improved by impoundage. Studies on Pickwick and Watts Bar reservoirs prior to and after formation indicate that pollution added to these lakes does not travel as far downstream as before. Fewer bacteria are found at Watts Bar Dam and in the lower end of the reservoir. Although the quantity of pollution added to the upper end of Pickwick Reservoir has increased during the past five years, its effect is not felt as far downstream, and the lower end of the lake is now suitable for all types of recreation.

Such improvements are expected due to the decreased velocities and the increased time of flow. Additional aeration from increased surface exposure has also aided the recovery processes. Turbidity has been reduced by sedimentation. Dilution ratios have been increased by stream regulation during periods of low flow. These factors were also observed during studies on Cherokee and Douglas reservoirs in 1943, and the preliminary studies show

that the reservoirs have remarkable abilities to reduce organic pollution.

Streams and lakes will continue to be used as a recognized means of sewage disposal. However, a fair overall point of view must be taken and this use of the waters should not be permitted to have precedent over other important uses.

Algae and plankton changes cannot be discussed in detail because relative information is not available for most of the Valley lakes. It was anticipated that such plant forms would cause some trouble to water supplies due to favorable conditions for increased rates of growth. Increases no doubt have been experienced, but with the exception of a few isolated cases, no widespread algae troubles have developed. Some troubles have been traced to local situations where industrial wastes aggravated conditions, but these were not attributed to the damming of the stream. Samples collected from Wilson and Pickwick reservoirs contained the normal flora with no unusual changes. Minor deviations may be necessary from previous water treatment methods, but the water plants should experience no serious difficulty. The benefits derived from the changes in stream conditions will outweigh the troublesome factors.

Population Distribution

Titles have been acquired to over 900,000 acres of land in the name of the United States for the Authority's activities and approximately 10,000 families have been relocated. The major portion of this has been in rural areas, but parts of villages and towns have also been affected. The problems involved have been numerous and urgent and in many instances of vital importance to the families and communities concerned.

Dr. Clarence L. Hodge, in his book entitled *The Tennessee Valley Authority* (3), says that legal obligations were fulfilled to the displaced families when the Authority purchased the land and the flowage rights, but that the Authority desires, as far as it is able, to assist such families and communities to make the wisest possible choice of new locations and the most advantageous expenditure of funds. A case history is maintained on each family until it is settled at its new location.

Almost 1,200 families moved from the land that was bought for Guntersville Reservoir in Alabama (4), and were in general relocated in counties in which they had formerly lived. About 800 were farm families of which only 130 were landowners. Upon relocation, 45 tenant families became owners of property. Employment by TVA on construction or reservoir clearance crews provided a helpful removal and readjustment resource. Some families were not satisfied with the move, as it meant leaving farms that had been in the family for generations, and some did not obtain land that was as satisfactory as before. Families were relocated at an average cost to the Authority of about $110 each, which does not include the purchase price of the land.

The Authority has also furnished technical assistance to community planning groups, and has taken every advantage of the opportunity to co-operate actively with the proper authorities (5). The Mayor and City Council of Guntersville authorized and appointed an official municipal planning commission, and the State Planning Commission and the Authority worked

with this group on the adjustments of the town of Guntersville (4). Other towns also followed this procedure of appointing a local planning commission.

In addition to this movement of families from the acquired land, large populations move into these areas in connection with the construction program. Local labor is used as far as possible, but it is necessary to draw on other sections of the country for skilled employees. These groups must be adequately housed in locations convenient to the construction.

The Tennessee Valley is predominantly rural, with only two cities of over 100,000 population and less than 30 additional cities of over 5,000 population in 1940. When nearby towns are accessible and of sufficient size to house the employees properly, villages are not provided by the Authority adjacent to the construction areas. If nearby resources are not suitable, the Authority provides villages near the project to augment existing accomodations. But in either event the facilities of the towns are taxed. Trailer camps and cabins are soon found scattered throughout a radius of many miles on every road creating many public health problems for the local, county and state health departments.

Experience has shown that time and considerable confusion are saved and the morale of the employee is improved when adequate provisions are made for housing prior to the actual construction of the dam. If adequate facilities are not provided near the construction site, the employee turnover rate will be enormous. There is nothing more discouraging than a situation where workmen literally have to fight for a place to sleep. It is economically impossible to provide all employees for this type of work with all of the comforts and conveniences of home, but the minimum standards at least can and should be maintained. Recently one of the project managers made the observation that he believed that "a dam is built in the camp as much as in the cofferdam." He was basing this observation on many years of experience at various construction projects where he observed the value of satisfactory housing and recreational facilities. A high morale results in a lower personnel turnover rate and in an increased rate of construction.

Health and Medical Care

The Authority provides medical care for all service-connected injuries. The size of the hospital at the project and the number of first aid stations depend upon local conditions. When the job is isolated and private medical facilities are not available to the workmen and their families, the Authority's medical center and medical care program are expanded to insure adequate medical service to those who live at the project. These services to the families are provided at a nominal charge.

Every effort is exerted to see that the employees work and live in an environment that conforms to best public health practices. Local, state and federal regulations serve as a minimum standard, and the advice and services of the best consultants are obtained. In matters of health the Authority, or any such agency, must recognize its two-fold obligation not only to protect the health of its employees, but to counteract any ill effects to the health of the community.

The Authority's Health and Safety Department has worked with and through existing health departments on matters of health, whenever possible, by the means of written contracts, joint committees and friendly personal relationships with officials and health authorities. The written contracts provide that the particular health department will furnish specified public health services for which the Authority agrees to pay a certain sum per year in monthly installments. Dr. Weintraub has included a sample of this contract in her book *Government Corporations and State Law* (6).

Malaria Control

W. G. Stromquist, when writing of malaria control in the Tennessee Valley in 1935 (7), said, "As the river is developed for navigation, flood control and the incidental production of power by the construction of dams, the resulting pools will produce a public health hazard if adequate malaria control measures are not provided for and maintained." The Tennessee Valley Authority realized this possibility. Detailed studies were undertaken by the construction, engineering, operation and health departments and as a result the bases or foundation of the control program against mosquito production has been built into the structures and operating schedules of the water control facilities (8).

"For intelligent planning and operation of malaria control measures, it is essential to know the distribution of malaria in the area affected. It is also important to determine the malaria incidence before and, from time to time, after impoundage so that the effect of impoundage and the effectiveness of control can be measured. The population affected is that living within one mile of the reservoir, the approximate flight range of the malaria-carrying mosquito" (7). Malaria may not be important in many sections, but this factor must be considered when artificial ponds or lakes are created in a malarious region. Surveys are made to determine the amount and type of infection. The foci of infection are located and a survey of all mosquito breeding places is made. These studies are made before and after impoundage by malariologists, entomologists and engineers.

Anopheles quadrimaculatus is the principal carrier of malaria in the Tennessee Valley and the southeastern part of the country. "Conditions favorable to *Anopheles* are constant water level; accumulations of fine flotage or debris and aquatic or semiaquatic vegetation which offers protection to the larvae; minimum wave action and absence of natural enemies of mosquito larvae. . . . The breeding occurs in shallow water along the shore, especially in the upper ends of inlets or tributaries of a reservoir, and is most extensive during the first few years after construction, before a biological balance is established" (7).

The Authority has realized its obligations to the Valley and has carried out an effective malaria control program with the greatest effort directed toward the elimination of mosquito breeding areas. Screening projects have been set up in many localities where it was found that the cost of eliminating the mosquito was prohibitive. In 1941 the Authority instituted a new attack on the elimination of breeding areas by "building them out" either by fills or diking and dewatering. Drainage had already

been practiced for many years. Cost estimates indicate a considerable saving in operation by these methods of eliminating breeding. Of even greater importance, they will provide more effective control.

During the *Anopheles* breeding season, larvae and mosquito catching stations are maintained in the areas adjacent to the reservoirs. These are inspected weekly and the system of reporting breeding has been developed to such a degree that it is possible to plan the attack week by week with larvacides being applied as needed. It must be remembered that the dams and reservoirs were built for multiple functions, but because the creation of artificial lakes provides conditions favorable for the prolific production of malarial mosquitoes, it is necessary to tie the malaria control program into the whole program of water control.

The Authority's malaria control starts with the planning of the dam. Each dam is provided with a "malaria surcharge," which permits the seasonal and periodic fluctuation of the water level (8). This is necessary to the maintenance of clean shore lines and other conditions inhibiting anopheline production. Before the reservoir is filled it is cleared to minimize flotage which provides excellent breeding places. No reservoirs are filled during the mosquito breeding season. When the final stage of operation is reached, fluctuations within the "malaria surcharge" are arranged through co-ordinated load dispatching in such a manner as to provide maximum values for mosquito control. Unforeseen events often prevent the operating schedule from being followed as well as desired, in which case oil and Paris green are applied as larvacides.

Silicosis

Pneumoconioses, such as silicosis, are a potential occupational hazard when dams are built in areas in which high silica-content rock occurs. Although the danger is confined to the men that work in proximity to the high concentrations of silica dust and not to the population in the area, it must nevertheless be considered as a condition created due to the construction activities. Every precaution must be taken to safeguard the workmen from this form of lung fibrosis which is caused by silicon dioxide, as quartz, inhalation.

This potential hazard has existed at the Authority dams in North Carolina where there are high silica-content rock formations (9). The dust is produced in the aggregate crushing operations, foundation preparation and tunneling. The tunneling operations also afford possibilities for toxic poisoning from gases contained in exhaust fumes from internal combustion engines.

The Authority has taken every precaution to combat these hazards. The workers for these jobs are selected for fitness by means of initial physical examinations and chest x-ray pictures, and periodic examinations are also made to insure fitness. Efforts are made to control the dust at its source. Segregation, enclosing the process, wetting the dust and exhaust ventilation are used as control measures. Individual protective respirators are necessary under some conditions. By a combination of these methods the workmen are protected from excessive exposure.

Recreation

Recreation in connection with the newly-formed lakes has received considerable publicity. Many feel that by

the development of the recreational potentialities of the region, those areas which have to a large extent been nonproductive may prove of economic benefit not only to a large number of permanent residents but also to the Valley as a whole (3). The Authority recognized this important "byproduct" of multiple-purpose dams and reservoirs and has established game refuges, picnic grounds, parks, bathing facilities and pleasure-boat docks. Many such developments also have been established in co-operation with public and private groups. As this development continues the responsibility is being assumed more and more by the state and local government. In many instances individuals have been licensed to operate concessions.

While it is desirable to promote recreational developments in the Valley, it must be recognized that definite public health problems may be created by concentrating recreational activities in limited areas unless proper planning and control are exercised. The Authority must assume the responsibility for the maintenance of adequate standards of sanitation and safety, although this responsibility may be shared with other co-operating agencies.

Regulations have been adopted by the Authority's Health and Safety Department to cover the operation of these facilities based upon the best present-day public health conceptions. The state and local health departments are co-operating in an effort to educate the operators as well as the public to an appreciation of sanitation and safety.

In general, the quality of the water in the lakes is now more suitable for swimming than before impoundage. Standards for bathing waters have been adopted, and areas are being approved on this basis. Bacteriological analyses are being correlated with sanitary surveys in arriving at workable standards. In the summer of 1942 a portable laboratory was placed in the field to make detailed analyses of water samples immediately after collection. This was an improvement over the previous arrangement whereby samples were shipped to the examining laboratory arriving the day after collection. It also offers an opportunity for close correlation between the analytical results and field observations.

Fishing and Hunting

Fishing and hunting are also becoming very popular around these new lakes. It is reported that the Wheeler National Wildlife refuge during the 1942–1943 winter and season of migration supported from 1,000 to 6,500 Canada geese and from 10,000 to more than 20,000 ducks.

Data obtained from an extensive fisheries census indicated that 4,750,000 lb. of fish were taken from five TVA reservoirs in 1940. This catch, including all types of fish, involved a total of more than 1,250,000 man-days of fishing (10). Incomplete information indicated that these figures are increasing each year.

Commercial fishing has shown a tremendous increase during the past few years. In Wilson Reservoir alone, between August 1941 and July 1942, 600,000 lb. of spoonbill were taken, representing an average income of $1,680 per fisherman for the eleven-month period. In northern Alabama, during the first six months of 1943, six fish buyers alone purchased more than 300,000 lb. of fish from local fishermen who were paid more than $62,000 for their efforts. This does not represent the total catch. Spoonbill, sturgeon and catfish represent the

major part of the commercial catch, although carp, buffalo and drum are also caught.

To encourage the utilization of these rough fish, the Commerce Department of the Authority is experimenting in processing fish by canning, smoking and freezing. Experiments have also been made on processing fish offal to obtain fish meal and valuable fish oil.

Food Supplies and Agriculture

That the quality and quantity of food have always affected health is recognized now more than ever because of current world conditions. Changes that affect food supplies are important. When dams are built and areas are flooded, many acres of land are inundated and can no longer be used for cultivation. The total effect on agriculture in the Tennessee Valley has been small, although it has been important to many individuals. This problem has been realized and the Authority, the Department of Agriculture and the State Agricultural Extension Services have aided these individuals in relocating in suitable areas.

The previous farm and forest practices in the Valley have left large sections of the lands partly or totally depleted of topsoil, resulting in submarginal or useless farms. In connection with the program for development of the region, the Authority and other interested agencies are carrying out extensive programs of soil enrichment and farm improvement to check soil exhaustion. Phosphate fertilizer produced by the TVA at Wilson Dam has been used for this farm test-demonstration work. In 1941 tests were under way on over 32,000 farms in 23 states (11). Terracing has been encouraged to aid in the control of erosion. Rural electrification is bringing electricity to the farm. Many farm practices have been encouraged that will contribute directly to better living and higher incomes. Experimental work has also been carried on in connection with food processing, especially quick-freezing and canning. Work has also been done on stock feed.

Industrial Developments

Industrial development in the Tennessee Valley may be considered an indirect result of the Authority's dams. Many of the new industries have been attracted by the extensive electric power facilities, while others have been attracted by other resources. Industrial development brings changes, many of which are related to public health.

The Tennessee Valley is predominantly rural, but if the area should become highly industrialized in certain sections, large groups of workmen and their families may be brought together near the industries. This could create public health problems, especially those associated with housing, water, sewerage, food, recreation, general health conditions, hospitalization, schools, etc. Many local and state planning groups are already formulating tentative plans in connection with postwar developments. Such industrial changes would raise the per capita income in the area and this should tend to raise the standard of living. Industrial waste disposal must also be considered for many of these industries. This must be appreciated by the authorities or serious stream pollution will result. An organized effort should be made to see that the industries locate at the most desirable points in the Valley to utilize resources and to protect present and future water supplies by treatment of wastes or proper disposal.

Safety

Organized safety programs have been carried out in connection with all of the Authority's activities, as this phase of public health is important to the individual as well as to the area. Poor safety practices, especially on large constructions, cause many accidents and injuries which often prove fatal. The Authority has made every effort to reduce these accidents and injuries through education of the individuals and the enforcement of safe practices of operation. The record has been good and many projects have been placed on the National Safety Council's honor roll for outstanding safety records.

Safety has also been stressed for recreational activities, especially those activities such as boating and swimming, which are new to the area and which have been directly or indirectly brought about by the Authority. The safety staff co-operates with existing agencies outside the Authority in establishing and developing sound policies and programs in the Valley region.

Health Education

Health education, which is important to the success of any public health program, is an integral part of the Authority's program of service and research. It is the means by which acquired health knowledge is transmitted to the individual. Public health is not achieved through progressive regulation but rather through intelligent public co-operation.

The Health and Safety Department has provided health guidance for the employees with examinations at yearly intervals, or more frequently if indicated. Employees are encouraged to take advantage of these facilities and to consult the medical officers with reference to health problems. All employees are immunized against smallpox and typhoid.

At the request of numerous agencies in the Valley states, the Health and Safety Department has participated in planning conferences and has co-operated in the development of health education programs and the preparation of educational material.

Water Supplies

The effects of the agency's dams and reservoirs on public and private water supplies have been numerous and varied. During the period of the construction of the dam and the preparation of the reservoir for flooding, a complete survey is made of each water supply that is located in or might in any way be connected with the future reservoir. Direct damage or damages that will result from the taking or flooding of property are obvious, and the necessary adjustment has often consisted of the reconstruction at another location of that part of the plant damaged, or perhaps making only minor changes in existing structures. At times, a complete change in the source of the supply with reconstruction of all but the distribution system is necessary. "The Authority has followed the practice of negotiating settlement contracts providing for the adjustment or relocation of affected structures in preference to exercising the right of eminent domain" (12). In some cases settlement has been made by cash payment, while in others the Authority has performed the work with its own forces.

Guntersville, Ala.

At Guntersville, Ala., the water supply was obtained from a spring in a

adjacent creek bottom at a point below the proposed Guntersville Reservoir level (13). This spring was often inundated by the Tennessee River when it was at flood stage. Investigation revealed that other springs in the area were not suitable for a supply capable of expansion to meet ultimate needs. Since the Tennessee River was the only available adequate supply, a complete treatment plant of 0.65-mgd. capacity was installed by the Authority. The total cost of this adjustment was about $117,700. The Authority also paid the city about $60,000 to set up a trust fund for equipment replacement and to cover increased operating costs of water and sewerage systems (4).

Decatur, Ala.

A cash settlement of about $55,000 was made to the Alabama Water Service Co. which supplies water for the city of Decatur, Ala. (14). The raw water pumping station was to be flooded by backwater from Wheeler Dam and there was a possibility of infiltration of river water into the clear water well at maximum pool elevations. The original plant was built about 1889. Some portions were obsolete or inefficient compared with present water treatment practices, so the water company elected to add an amount to the cash settlement and to reconstruct the entire plant, with the exception of the recently constructed sedimentation basin, along more modern lines.

Sheffield, Ala.

Sheffield, Ala., is located at the upper end of Pickwick Reservoir, and downstream from the city of Florence and activities in the vicinity of Wilson Dam. Domestic sewage and industrial wastes are discharged to the reservoir upstream from the water plant intake. This water plant was old and plans had been discussed for the construction of a new plant prior to the creation of Pickwick Reservoir. The existing plant was subject to flooding and this danger was increased by the backwater from the dam. It was also claimed that the impoundage would increase the pollution hazard, as one of the city's principal outfall sewers was located only a few hundred feet from the intake. "The city of Sheffield and the Authority reached an agreement whereby, upon payment of $10,000, the Authority was relieved of any liability" (15). The city, with the aid of the Works Progress Administration, then built a new water treatment plant.

At that time there was but meager information regarding the changes that could be expected in the quality of the river water following impoundage. Complete chemical and bacteriological analyses were made of the water at the intake for a year prior to impoundage (1) and these studies have been continued since the lake was filled. No increase has occurred in the pollutional load at the intake attributable to the Authority's activities in changing this flowing stream to a relatively constant level pool.

Kingston, Tenn.

A spring, the source of the Kingston, Tenn., water supply, was to be flooded by the Watts Bar Reservoir. This town is located at the confluence of the Clinch and Tennessee rivers, and, although it was felt that a filtered river water supply would be the best permanent solution, previous stream studies and experiences at a filter plant downstream cast doubt on the advisability of using either of these two rivers. The Emory River contained

waste from a paper mill and discharged into the Clinch River near its mouth, while the Tennessee River contained large amounts of domestic sewage and industrial wastes. This concern was later borne out by experiences following impoundage when the black water of the Emory River was found to travel many miles up the Clinch River beyond the proposed intake location.

The solution for Kingston was the development of a spring several miles from town from which water flows by gravity to a new pumping station in the town. The town also considered this more desirable as the filter plant would have been more difficult to operate. The entire cost of these changes was paid by the Authority.

Harriman, Tenn.

Harriman, Tenn., on the Emory River and in the backwater of Watts Bar Reservoir, is having considerable trouble treating the river water for domestic use, as at times the water is polluted with industrial wastes and sewage from a paper mill, two textile mills and the city sewers which enter below the water plant intake. Under certain stream, reservoir and weather conditions upstream currents carry these materials for more than a mile to the water plant intake. Complete studies have been made of the stream and the wastes, and density currents, which exist at times in this arm of the lake and are responsible for the upstream direction of flow, have been discovered. An economical and satisfactory method has not yet been found for conditioning this water at the filter plant. It can be made bacteriologically safe but not potable at all times.

The TVA Legal Department has expressed the opinion that since the Emory River is a navigable stream, the federal government has no liability in connection with improvements of the stream for changing either the quality or quantity of levels of water flowing in such streams. The pollution is discharged to the streams by concerns which cannot acquire a right to use the waters of a navigable stream. The Authority is not responsible for this pollution and without violation of established policies cannot build structures to correct such conditions. However, every aid is being given to both the water company and the paper mill, which is responsible for the major pollution, toward reaching a satisfactory solution. The Authority has also regulated the level of the Watts Bar Reservoir to prevent the formation of these density currents, even though such regulation interferes with other Authority operations. At present, Harriman and the paper mill are confronted with three alternatives: (1) to build a diversion dam and new intake for the water plant; (2) to carry the waste far enough downstream or out of this drainage basin where it will not create further nuisance at Harriman; or (3) to treat the waste so that it will no longer be objectionable. Although any of the three plans will relieve the problem for the water plant, only the last would improve the stream conditions in Watts Bar Reservoir. It is also important that only the first would guarantee protection to the water supply under all operating conditions.

The paper mill has obtained a 90,000-gal. steel barge and a tug boat at an initial cost of $30,000. Digester waste is stored in a dirt pit and then pumped into the barge and carried about 15 mi. downstream and discharged into the main body of the reservoir. At present five trips are made each week. This has removed a

considerable portion of the waste, but at times the water still cannot be treated satisfactorily. The alternatives are again being studied and the new intake is being given consideration. The paper mill is also reported to be studying methods of waste disposal.

A serious public health problem exists at Harriman. Although the state health department has assured the city that the water supply is safe, many people prefer to use untreated well and spring waters of questionable bacterial quality because of the color that is at times present in the city supply.

Knoxville, Tenn.

The Holston and French Broad rivers join above Knoxville, Tenn., to form the Tennessee River. Both of these tributaries are heavily polluted with sewage and industrial wastes from towns and industries on the two watersheds. The intake for the Knoxville Water Works is located a few miles downstream from their confluence, and the wastes carried by the streams have caused considerable coagulation troubles combined with tastes, odors and color, especially during periods of low stream flow. These two tributaries are now regulated by storage dams, and, under certain conditions of regulation, additional difficulties might be experienced. The Fort Loudoun Dam has been constructed downstream from Knoxville, providing a pool into which the sewage and wastes from Knoxville are discharged. Studies are being made of the three reservoirs and the conditions at the water plant are being carefully observed, but conclusions cannot be drawn on reservoir conditions until at least another season when the full effect can be observed under various methods of reservoir operation. Recent experiences with the uncanny ability of pollution to find its way *upstream,* apparently against the direction of stream flow, indicate the necessity for careful study of this section of the river as the reservoirs are formed.

Fortunately, Knoxville is located on the main body of the lake and the sewage and industrial wastes will have a much higher dilution ratio than would be experienced on small streams entering arms of the lake. Extremely low stream flows should not exist, as one of the chief purposes of the upstream dams, Cherokee and Douglas, is to increase low flows for navigation and power. These factors will tend to improve conditions and aid stream recovery, but they are dependent upon proper regulation.

Charleston, Tenn.

At Charleston, Tenn., the town water supply well is located about 50 ft. from the steep bank of the Hiwassee River. As the river level was to be raised by backwater from Chickamauga Dam and would be above the elevation of the bottom of the well, concern was felt over the possibility that this change might cause river water to seep through to the well. Investigations failed to locate a more suitable supply, or the same spring at a higher level. Studies were made of the river and well water under various conditions for a period of two years, and an intensive eight-month study was made at the time the reservoir was filling. Bacteriological examinations indicated that there was no evidence of the lake affecting the well. There was no increase in turbidity or chlorine demand following the filling of the reservoir, and there was a positive head of water in the well at all times.

Cherokee Reservoir on the Holston River will be subject to a maximum

drawdown of 95 ft. from the maximum high water level during dry seasons when its stored water is released to supplement flow in the Tennessee River. A new water supply intake was built for Morristown, Tenn., so that water could be pumped at all reservoir levels. Duplicate pumps were provided instead of the previous single unit.

Jefferson City, Tenn.

The spring from which Jefferson City, Tenn., obtained its water supply was to be flooded by the Cherokee Reservoir. Studies over a period of a year indicated that the only suitable substitute spring became turbid at times and that the water was harder. Analyses of the two springs showed average turbidities of near 0 and 19 ppm.; color, 0 and 9 ppm.; and hardness, 206 and 223 ppm. Increases were not excessive. A softening plant was constructed, however, and arrangements were made for its operation. Recent check analyses indicate a turbidity of near zero and a hardness of about 64 ppm. for the treated water.

It has also been necessary to consider private wells and springs located near the reservoirs. All complaints are investigated by the various Authority departments and the findings are correlated and sent to the legal department. At Waterloo, Ala., complete studies were not made of such supplies prior to filling the Pickwick Reservoir. As soon as the lake was formed, complaints were received from a group that the lake had ruined their wells. Surveys indicated that the water surface elevations in the wells, and in many cases the bottom elevations also, were above the lake level. Undoubtedly the ground water table had been raised by the lake and also by the heavy rains which were experienced during this period. The turbidity probably resulted from surface seepage caused by the rains, for the wells later returned to normal. Histories revealed that many of these wells were polluted before this change. Studies prior to impoundage would have proven many of these points and avoided considerable confusion.

Soddy, Tenn.

Similar experiences were anticipated at Soddy, Tenn., on Chickamauga Reservoir and thorough studies of the ground water table were made. These were supplemented by chemical and bacteriological examinations which revealed that all of the ground water was heavily polluted prior to impoundage. Chemical analyses were not correlated with other criteria as well as expected due to the variety of rock formations in this region. These studies served the dual purpose of providing valuable factual information for the Authority and of establishing confidence in the minds of the local people that the Authority was interested in their welfare. Such studies of ground water are now standard practice during the period of reservoir preparation and an effort is made to explain pollution of wells to the owners.

The Authority has constructed and operated water supplies at each of its projects. These are designed and operated according to the best accepted public health practices. Plans are submitted to the respective state health departments for approval prior to construction, and the state health departments are kept informed of operations through copies of the monthly operation reports and weekly bacteriological examinations. A high standard of bacterial quality is maintained for these

supplies. During the past five years less than 1 per cent of the finished water samples have contained organisms of the coliform group.

A procedure for batch chlorination of water used by isolated groups of employees has also been followed, and residual chlorine is maintained in these supplies at all times. It is believed that no disease has had its origin in any of the Authority's water supplies.

Sewerage

Proper and adequate excreta disposal is of vital importance to the health and welfare of the people in an area. Pollutional materials discharged into streams may render the water unsuitable for a water supply or for recreational or other uses. The Authority, conscious of these facts, has studied the effects of the program on sewage disposal and sewerage facilities.

Direct damage from flooding creates an engineering problem of adjustments to the affected systems. At many points it has required only the relocation of the outfall sewer, while at others it has required complete reconstruction of the treatment plants at higher elevations with pumping of part or all of the sewage. The Authority's activities have had no great effects on sewerage facilities in the Tennessee Valley.

The Authority constructed intercepting sewers at Decatur, Ala., to convey the sewage to the river downstream from the city, and two pumping stations to lift the sewage to the outfall sewers. Before Wheeler Dam was built, the Decatur sewage was discharged through six outlets to the river without treatment. These were to be flooded. The total cost of reconstruction was about $207,500, which included $25,000 paid to the city in preference to the construction of a sewage treatment plant and to release the Authority from damage after sewer construction (14). The Authority also agreed to furnish power for the operation of the electric pumps.

At Dayton, Tenn., the Authority constructed a primary-treatment sewage disposal plant to replace the two old plants which were submerged by Chickamauga Reservoir. The complete sewage works adjustment cost about $123,000 and consisted of intercepting sewers, a pumping station to raise the sewage to the plant and a $37,000 disposal plant.

It was necessary to construct a dike at Guntersville to protect the industrial section of the town. The total cost of this shore line protection was about $323,000. Considerable relocation was also required for the sanitary and storm sewerage systems. These adjustments consisted of the construction of a high-level intercepting sewer, a low-level intercepting sewer, outfall sewers, a combined sanitary and storm sewage pumping station and a force main. The total cost for relocating the sewerage system was about $266,000. A joint trust fund of $60,000 was set up for equipment replacement and to cover increased operating costs of the water and sewerage systems (4).

Outfall sewers have been relocated at a number of municipalities so that their operation would be satisfactory at the new reservoirs' water levels.

Most of the cities and industries in the basin discharge their sewage and wastes to the streams untreated. It is estimated that in 1940 approximately 24 per cent of the Tennessee Valley's population was connected to sewers and that only 19.7 per cent of the sewage from this connected population received any form of treatment. The over-all reduction in sewage population

TABLE 1

Final Costs of Land and Land Rights, Dollars

Item	Projects				
	Wheeler	Guntersville	Pickwick Landing	Chickamauga	Norris
Total costs of project	30,378,889	33,188,040	31,841,229	35,671,127	32,269,027
Purchase price of land	4,241,137	5,552,132	3,324,316	5,670,179	7,648,272
Expense of land and privilege acquisition	523,500	884,951	560,178	819,915	951,857
Relocation of highways and bridges	1,128,039	2,827,292	1,166,961	2,358,096	2,689,700
Relocation of railroads and bridges	231,868	336,223	514,609	6,071	1,177,693
Relocation of other structures and improvements	179,138	805,077	198,081	350,518	364,159
Protection of existing structures and improvements	515,662	447,331	10,168	—	81,075

equivalent due to treatment has been calculated to be about 7.5 per cent (2). Only a small percentage of the industrial wastes receive any form of treatment. At many points these discharges create bad stream conditions, and this is being recognized. At the present time the Authority is definitely limited in the remedial measures that it can use. This pollution and its effects have been investigated by the TVA and the various state health departments, and aid is being given in the locating of new industries, so that their wastes will have a minimum effect on the water supplies and recreational areas.

The construction camps and villages of the Authority have been provided with adequate sewage treatment depending upon local needs. Norris village has complete treatment because the effluent is discharged into a small stream which flows to the Clinch River, and adequate dilution is not provided. As Chickamauga Dam is located upstream from the Chattanooga water supply intake, primary treatment and chlorination were used for the construction camp. Many of the construction camps and villages are located on polluted streams or in isolated areas and therefore require no sewage treatment during the relatively short period of construction, as the effect on the stream is very small. However, at least primary treatment is provided for the permanent installations.

Studies at the present time indicate that the streams that will be dammed in the Tennessee Valley will continue to serve as a means of sewage disposal with at least their present efficiency. Large bodies of water are being created providing sedimentation and aeration, and the low summer stream flows of previous years will no longer aggravate stream conditions. Additional water will be provided from tributary storage reservoirs, and this will aid the stream in its physical and biological recovery processes.

Cost of Protecting Existing Structures

Table 1 compares the total cost of the various projects with the cost of the land and the costs for relocating and protecting existing structures along the reservoirs. The costs for protecting water and sewerage systems, city streets, city bridges, etc., are included in the item "Protection of existing structures and improvements." The item "Relocation of other structures and improvements" includes power lines, telephone and telegraph lines, grave removal, relocating families, preservation of prehistoric material and relocation of existing water and sewerage systems (4, 14, 15, 16, 17).

Summary

The Tennessee Valley Authority's program to transform a flowing stream into a series of lakes has resulted in many social, economic and physical changes of public health significance. These effects on public health are easily recognized but difficult to measure quantitatively, because there are so many indeterminate factors which must be considered. At times environmental changes may be determined by measurements or laboratory analyses, and in these cases results can be presented as evidence.

This pioneer endeavor is comparatively young and the changes in the area have just taken place, or are still in the transition period. Observations on many phases of public health, in order to be conclusive, must be of long duration with adequate initial data and good controls. In many cases these requirements are not yet met, and the conclusions therefore are based upon the best available data.

About 900,000 acres of land have been acquired and about 10,000 families have been relocated in connection with the preparation of the reservoirs. This movement of population, along with the movements of workmen into the construction areas, has presented many public health problems for the Authority as well as local and state health departments. These conditions have been studied and every effort has been exerted to maintain proper standards for environmental sanitation. No permanent ill effects are apparently felt by the local communities due to the Authority's activities. During the peak of construction facilities were overcrowded, but the financial gain was sufficient to offset the temporary inconvenience.

The potential public health hazard of malaria was realized from the beginning and permanent control measures have been built into the structure and operating schedules of the water control facilities. Extensive studies have been carried out in determining the most effective control procedures. Surveys indicate a reduction in the malaria morbidity rate in the area affected by the lakes.

Silicosis is another potential hazard that has been considered at the dams that have been built in areas in which high silica-content rocks occur. Other occupational disease hazards have also been studied in the industrial operations of the Authority. These studies and corrective recommendations have been made by the Industrial Hygiene Section.

Recreational facilities are rapidly developing on the shores of the newly-formed lakes. Appreciating the advantages and benefits to the area, an effort is being made to see that these follow best public health practices.

The water supplies of the Valley are very important to the communities they serve. If the structures were to be flooded, they were adjusted or relocated. When the quality of the water was affected, the remedy was usually not as simple, and at times a complete change in the source of supply or treatment was necessary. Damages to sewerage facilities were not great and adjustments usually consisted of simply relocating some of the sewers and outfalls.

Water supplies and sewerage facilities are affected by the general stream conditions. Many physical changes are taking place in the stream as the nine river dams on the Tennessee River are nearing completion, and storage reservoirs are being provided on the main tributaries. The Tennessee Valley as a whole contains relatively clean streams when compared with other regions in the United States. A few changes have taken place since impoundage which may have resulted in adverse local conditions, but the streams themselves are in a better condition from the standpoint of quality and quantity. A more accurate over-all picture of the changes that have taken place in the quality of the water is desirable, but complete information is not yet available for all sections.

The Authority has encouraged and co-operated with existing local, state and federal agencies and organizations in problems of public health, agriculture, industrial development, regional planning and safety. The work of existing agencies is not duplicated, but the effort is directed toward the joint solution of new and existing problems.

The Health and Safety Department of the Tennessee Valley Authority has realized the obligations of the Authority on matters of public health. The Authority must not only protect the health of its employees, but must also counteract any ill effects to the health of the community caused by activities associated with the building of the dams and the formation of the reservoirs.

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Speaking of Flood Control

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Too Many Plans and Too Many Dams Do More Harm Than Good

About sixty years ago in the United States of America we started draining our marshes, sloughs and small lakes and hurried the natural rainfall from the uplands off into the rivers to the sea. Final results: disastrous floods on the lowlands and aggravated droughts and lowered ground water tables on the agricultural plains. (It is reminiscent of the late William Allen White's eloquent diagnosis of what was the matter with Kansas: It started raising hell and later suffered from over-production.)

Experience during the last twelve years with floods and droughts of increasing violence have driven home the folly of over-drainage excesses. It has ruined millions of acres of productive land. Now, reversing the process, we are threatened with a veritable epidemic of dams. As if dams were the magic patent medicine to cure all the ills of our wasting natural resources, the Congressional calendar is full of measures to authorize the building of an amazing number of dams, at tremendous cost.

Too many dams may be just as destructive as too much drainage. Our great rivers and their tributaries are to this continent what the jugular vein, arteries and veins are to the human body. They are its life blood. Major operations on the blood stream of our continent should be undertaken only after the most careful and rigid diagnosis. Too many tourniquets on our arteries may be just as unhealthy as too much bleeding, and too many dams may be just as destructive as too much drainage.

No one will deny that we need to hold the waters back on the upland, ameliorate the floods, water the agricultural regions and to some extent we may be able to use the byproducts of water management for the production of hydro-electric power and river navigation. It sounds like a good idea. But having had one sad experience in trying to improve nature's mechanism, it might be wise to look before we leap into another bramble bush, the benefits of which are thus far somewhat speculative.

Let it be said in the beginning that this discussion is not intended as an argument against flood control, hydro-electric power dams or any of the other advertised objectives which may be beneficial, but is an attempt to aid in the discrimination between effective engineering projects with broad public benefits and those ill-starred promotional schemes which will cost a great deal of money but which are likely to prove ineffective and generally disap-

pointing in their consequences. We are going to have flood control and hydro-electric power dams for the very good reason that we can't get along without them. We should, by all means, hold the rains and melting snows on the uplands, preferably by storage within the soil itself, for the sake of continued agricultural production. We need to prevent soil erosion as one of the greatest of economic factors in the future welfare of our country. Reforestation is one of the first aids in flood control. Recreation and wild life are among the normal byproducts of environmental restoration. All of these variegated objectives are inextricably associated with what is called our national water management program. The danger lies in the fact that in our hysterical haste we may not do the job wisely.

At the present moment there are pending for Congressional consideration three wholly separate and uncoordinated water management programs, each damming the same waters, backed by three separate departments of the federal government and supported by three distinct pressure groups. Each program has been drawn in complete disregard of the other two.

The three government agencies are the Army engineers, with their flood control program; the Reclamation Service of the Department of the Interior, with its irrigation program, and the President of the United States, with his extension of the Tennessee Valley Authority principle to seven of the major river basins of the continent. Each one of the three programs is of sufficient magnitude to challenge the imagination of a nation determined to outwit economic stagnation in the post-war era. All of them together present such a picture of duplication, overlapping and irreconcilable conflict with natural conditions as to cripple seriously each other's effectiveness.

Three cooks trying to peel one potato couldn't possibly get more in each other's way.

Apparently none of the proponents has remembered that dams built in a muddy stream will be choked with silt before they can pay for the cost of construction. All of the agencies overlook the impracticability of attempting to combine the production of hydro-electric power and flood control in the same dam. In none of the programs have they allowed for the fact that fish, wild life and recreation are very largely obliterated from the fluctuating waters impounded by flood control or power dams.

Soil erosion, which constitutes at the same time one of our greatest economic problems and the No. 1 menace to any and all water management programs, is left until the last, as if it might take care of itself in some mysterious manner not provided by the engineers.

Overlooking for the moment the lack of co-ordination in the three proposed water management programs, it would be a serious oversight to proceed on the theory that all dams are inherently beneficial to all public interests, or that the more dams we have the richer life will be in this nation. Too many dams, built without adequate consideration of their destructive as well as their beneficial consequences, could be as wasteful of natural resources as too many drainage ditches.

A scientific approach to any water management program would dictate that no heavy silt-bearing stream be dammed until soil erosion had been effectively controlled.

The great Missouri River drainage basin is a fair case in point. This river

and most of its tributaries, from its headwaters in the Yellowstone Park region to its junction with the Mississippi River, meanders through the soft loess plains and alluvial soils of nine agricultural states of the Middle West. Its waters, even in seasons of slack rainfall, are perennially opaque with a heavy load of silt. Its nickname is "The Big Muddy." Leaving out the extremes of exceptional flood conditions, its average content of solid material runs from 520 to 642 units a million. A dam built across the Missouri River anywhere between Sioux City, Iowa, and Yankton, S.D., would fill up with mud at the rate of six million (6,000,000) tons of solid matter each year. This siltation hazard prevails without serious variation throughout the entire Missouri River drainage basin and its tributaries, the amount of silt being relative to the size of the stream. Building dams under such conditions is of course a questionable procedure.

Muddy waters hold the silt in suspension only so long as the current in the river maintains its speed of flow. Silt carried by river currents is exactly comparable to dust blown by the wind. As soon as the wind stops blowing the dust settles to the ground. Just so when a river current strikes the quiet waters of the lake back of a dam its current comes to a standstill, the heavier particles of eroded soil settle to the lake bottom and remain there. Even the recently developed roller-type dam, which allows the water to escape from the base of the dam instead of over the top of the spillway, only scours the mud from the bottom of the impounded lake as far back as the eddy of the dam. A hundred yards is the practical limit.

The productive efficiency of any dam, whether for the generation of electric power or for the storage of emergency flood waters, is wholly dependent on the volume of water it can hold in its "mill pond." Any mill pond which loses the equivalent of 6,000,000 tons (approximately 6,000,000 cu.yd.) each year would be of doubtful expediency.

An annual blanket of silt deposited on the bottom of any lake, artificial or natural, will have far-reaching consequences on natural resources far in excess of the direct effect of diminishing power production or flood control.

Why the Frantic Haste to Rush Through Extravagant Programs?

One of the lures held before the public is the inviting prospect of a large and presumably beautiful lake, full of fish for the fishermen and providing facilities for bathing, boating and outdoor recreation. Several things take place which would seriously mar all of these pleasant prospects in any artificial lake created by a dam in the Missouri River drainage basin:

(1) The silt deposited annually on the bottom of the lake smothers all aquatic vegetation.

(2) As vegetation finally disappears, fish populations fall off rapidly and at last disappear.

(3) When both vegetation and fish life have gone, the natural foods sought by migratory water-fowl no longer invite their presence.

(4) Other forms of recreation, such as bathing and boating, are rendered uninviting by a lake the shores and bottom of which are lined with a thick, slippery coating of mud.

(5) No amount of local restocking with fish will maintain a fish population

in waters thus devoid of essential food and environmental conditions.

(6) If the above conditions are aggravated by violent fluctuations of the water levels, as is the case in all flood-control dams and most power dams, an additional hazard is added to destroy the public's enjoyment of the lake.

(7) Lowering the lake levels draws the water away from the normal shoreline, leaving a broad expanse of mud-covered beach between the dry land and the lake waters. Any one who has ever tried to land a picnic party on such a shore can testify to its lack of recreational qualifications.

A considerable amount of fiction has crept into the promotional oratory of the proponents of these flood-control, electric power and irrigation projects. They all claim that their dams will be all things to all men. Local chambers of commerce are, as a rule, captivated by magnificant dreams of the future development of great industrial centers, cheap electric power, a resort lake with fishing, hunting, boating and recreation, plus the expenditure of several million dollars for construction in their region.

One is led to wonder whether it is ignorance or deliberate deception which has prompted the use of so many false claims. One of the more glaring of the prevalent inconsistencies is the proposal by the Army engineers to maintain an empty reservoir to catch the flood waters and in the same breath to produce a commercial volume of electric current which necessitates a full head of water back of the dam.

No dam can be effective in the retarding of flood waters if the reservoir back of the dam is not kept practically empty in readiness to impound the unpredictable arrival of the next flood stage. It is difficult to conceive an industrial development which could subsist on electric power produced only during flood stages of the river. And yet the Army engineers' flood-control program, now being debated in Congress under the title of the "Rivers and Harbors Bill," includes many such irreconcilable claims. Even more astonishingly, it presents no aspect even remotely associated with the curse of siltation or its moronic parent, soil erosion.

It proposes to attack the problem of flood control by building large dams across the path of rushing waters after they have accumulated the full momentum of their combined forces. Holding the flood waters in the main body of the Missouri River is like trying to harness stampeding wild horses in full flight.

The Reclamation Service program recommends the more practical method of catching the wild horses on the prairies before they bunch up and start running away. It proposes to build more but smaller dams at the headwaters and upper reaches of the tributaries, and to use the impounded waters for irrigation of surrounding agricultural lands. It is an excellent beginning, if accompanied by erosion control on the lands above the dams. Without the latter it would be difficult to justify all their contingent claims of hydro-electric power production, recreation and wild life conservation. Violent fluctuations of the water levels are inherent in irrigation storage reservoirs, with destructive consequences already cited.

President Roosevelt's "Missouri Valley Authority" and the six other similar projects which he suggests, for a total of seven duplications of the TVA, form the only program yet proposed which has within it a consistently

comprehensive water management mechanism.

Theoretically, the MVA includes all the essential objectives: soil conservation, forestation, irrigation, navigation, recreation and hydro-electric power generation, with particular emphasis on the latter item. The mechanical possibilities offer an interesting subject for study, but it must be approached with some factors in mind for which the Tennessee Valley project provides no precedents. Compared to the Tennessee Valley terrain, the Missouri River basin covers a much greater area, and the hazards of siltation will be many times multiplied, with all the attendant liabilities.

If, as a preliminary to the MVA, a thoroughly engineered program of erosion control were established throughout the water management area before the dams were built, it would be difficult to find fault with its mechanics. The social and political aspects of the TVA methods, involving the centralization of authority in Washington over such a vast area and the consequent usurpation of all state jurisdiction, are quite apart from this discussion of water management and will merit attention and undoubtedly get it. It is with something of a shock one learns that the Department of Agriculture has not been consulted or been allowed the courtesy of reviewing these extensive national projects which intimately affect the lands and waters essential to our national food supply. At last reports, only the Reclamation Service had submitted its program to the Fish and Wildlife Service for expert advice on the biological aspects which might be encountered.

Obviously the frantic haste to rush through such extravagant programs smacks too much of a "gold rush" instead of "water management" to justify the monumental expenditures involved. It is a strange commentary on our so-called "planned economy." Students of conservation of our natural resources may well take counsel together for such remedial measures as they may discover and local chambers of commerce might well call in some experts to examine the river projects, if not their own heads.

This article first appeared in the January 30 and 31 issues of the New York Herald Tribune and because of its interest to the water works field has been reproduced above.

Stream Pollution in Tennessee

Report on a Study of Pollution of Streams and Other Surface Waters in the State and Program of Control

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EDITORIAL NOTE

THE material included in this report was based on records of studies made by members of the Division of Sanitary Engineering, information from published and unpublished reports of the Health and Safety Department of the Tennessee Valley Authority, the Department of Conservation, the U.S. Public Health Service and other co-operating agencies.

Roy J. Morton, Associate Professor of Sanitary Engineering, Vanderbilt University, was employed by the board as Consultant, and S. Leary Jones, assigned to the board by the TVA, was primarily responsible for assembling the material and preparing the report.

This carefully prepared 48-page brochure includes a preface; the conclusions arrived at by the board; a summary and review of the over-all picture, including tables showing population distribution, water and sewerage facilities, and sources of pollution with areas and populations affected; and recommended legislation. Following the body of the report are a comprehensive bibliography and charts and maps of Tennessee which illustrate the pollution in the various watersheds.

On Jan. 20, 1945, the report was submitted to the Hon. Jim Nance McCord, Governor of Tennessee, in compliance with the resolution passed by the General Assembly of 1943. The report is planned as a guide to the Tennessee General Assembly.

Certain portions of this report are considered to be of such general interest that they are reprinted below.

CONCLUSIONS

THE Stream Pollution Study Board has assembled the available data concerning the pollution of surface waters in Tennessee as authorized by the 1943 General Assembly and as instructed by Governor Prentice Cooper. The sources of pollution have been located, and the condition of the streams has been described in this report. The value and use of these waters have been reviewed, and the effects of the pollution on the uses of the streams have been investigated.

The board has reviewed stream pollution control programs and legislation of other states. Based on these findings and considering the conditions in Tennessee, the Stream Pollution Study Board recommends the establishment of a permanent stream pollution con-

trol board and an administrative state agency authorized, and provided with facilities, to carry on the activities essential for a well organized and effective control program. Legislation based upon sound principles controlling the discharge of pollutional materials is necessary, but pollution abatement and the protection of surface waters of the state depend perhaps even more upon competent administration. The water policy adopted by the state, and the effectiveness with which this natural resource is conserved will exert an important influence on the future development of Tennessee.

In 1940, the population of Tennessee was 2,915,841. Of this total, it is estimated that a population of 936,270 is connected to sewerage systems and discharges a total of about 100 mil.gal. of municipal sewage to the streams each day. Sewage treatment plants are provided for about 17 per cent of this sewage which reduces the pollutional load about 7.4 per cent.

It is calculated that the organic industrial wastes have a pollutional effect on the streams equivalent to the domestic sewage from a population of about 1,267,820. This is approximately 1.35 times the pollution caused by the domestic sewage in the state. This does not include certain inorganic components of industrial wastes which are toxic to fish or humans, impair domestic or industrial water supplies, or are objectionable for other reasons and prevent normal use of the surface waters. Between 200 and 400 mil.gal. of industrial wastes are discharged each day to streams within Tennessee. Very little is accomplished in the way of correction or treatment of these industrial wastes as the estimated reduction by treatment is only about 1 per cent.

There are approximately 15,000 mi. of streams and rivers that were considered large enough for classification as surface waters and of this total about 710 mi., or 5 per cent, are known to be seriously affected by stream pollution. At least an additional 850 mi., or 6 per cent, are damaged for many desirable uses and are not suitable for most recreational purposes. In reality, more than 11 per cent of the major streams are seriously or moderately polluted. For example, the Cumberland River within Tennessee is 311 mi. long. Approximately 15 per cent of this mileage is seriously polluted and an additional 40 per cent is moderately polluted. The Tennessee River and its two principal tributaries, the Holston and French Broad rivers, in East Tennessee contain 542 mi. of main rivers. Approximately 26 per cent of this mileage is seriously polluted and an additional 28 per cent is moderately polluted.

Surface waters have many important values which include their use for public and industrial water supplies, fish and other aquatic life, recreation, agriculture, power, navigation, esthetic values, and sewage and waste disposal. In so far as possible, streams should be maintained in conditions suitable for these uses. In many cases, the benefits from these uses cannot be determined in monetary values but, without surface waters in a usable condition, it is known that progress in Tennessee would be retarded and eventually cease.

There are 56 public water supplies in the state that use surface waters as sources of supply and furnish water to a population of nearly 600,000 people. This is approximately 20 per cent of Tennessee's total population. This use is increasing each year and if the

streams that are necessary as sources of domestic water supply are not protected, the health of many citizens of the state will be endangered. In 1940, the major industries when surveyed reported the use of nearly 200 mgd. of surface waters. At present, this use is probably nearer 300 mgd. Many sections of Tennessee rivers are already unsuitable for sources of industrial water supplies as the quality of water required for industrial use is often more exacting than for municipal use. Industries cannot locate in areas that do not have suitable water supplies.

Almost 120,000 fishing and hunting licenses were sold in 1943. Conservative estimates show that each fisherman spends a minimum of $10.00 per year which indicates that fishing is now a million dollar business. It is also estimated that potential sport fishing in Tennessee represents an annual expenditure of at least $5,000,000. Commercial fishing is becoming important in the state in the new reservoirs of the Tennessee River. The wholesale destruction of sport and commercial fish in Tennessee streams by pollution as occurred during the summer of 1944 in the Cumberland River, the Holston River and many of the smaller streams of the state is an economic as well as an enjoyment loss to the people of the state. This destruction of fish is only the visual evidence of unseen conditions that result in even more damage through continuous prevention of propagation and development of far greater numbers of fish.

Many new recreational sites are being developed each year on Tennessee streams but their recreational values often are depreciated or fail to materialize due to water pollution. Recreation holds promise of becoming larger than any other industrial or business asset in the region, for in 1941 it was estimated that recreational travelers spent over $104,000,000 in the state.

The Stream Pollution Study Board is aware of the necessity for using surface waters as the final disposal medium for liquid wastes from municipalities and industries. In considering this use of surface waters, the economic value of the municipalities and industries must be appreciated. Each is dependent upon the other for progress and both are essential to the fullest development of the state. The value of the streams as a recipient of the sewage and industrial wastes may at times be found to exceed their value for other uses, particularly when adequate water supplies can be obtained elsewhere. The board believes that it is of the greatest importance that a balance be maintained so that all of the essential uses in each area will be protected.

In most cases, the required protection of streams must be provided by sewage and waste treatment. Although complete treatment of all sources of waste will not be necessary, an adequate long-range program should expect to provide the requisite degree of treatment for the various sources of waste on each particular stream to protect its condition and essential uses. Available data are not sufficient to allow accurate estimates of the construction costs of such a corrective program but preliminary estimates indicate that they will be in the neighborhood of $50,000,000 for the entire state. The exact way in which these costs are distributed between municipalities and industries will depend upon the extent to which industrial wastes are disposed of in conjunction with sewage in municipal treatment plants.

Although the primary purpose of the Stream Pollution Study Board is to locate the sources of pollution and determine the condition of surface waters in Tennessee, the board feels that, in light of its detailed studies, it is warranted in recommending a type of legislation, administrative structures and controlling principles to correct the disabilities that are pointed out in this report and to prevent additional unnecessary pollution.

This study board recommends legislation which will place the responsibility for water pollution control in a Stream Pollution Control Board. It is suggested that this control board be composed of five members: the Commissioners of the State Departments of Health, Conservation and Agriculture to represent the state, and two appointive members to represent municipalities and industries, respectively. The administration of this program should be centralized in one state agency—the Department of Public Health—which would be authorized to integrate the efforts of all state and local agencies in order to avoid conflict and duplication and which would carry out the administrative duties of the board.

The Stream Pollution Control Board should be given the following powers:

1. Power to define what constitutes pollution, with the definition based on the consideration of all phases of water use.
2. Power to promulgate rules and regulations to interpret and facilitate its authorized powers and functions.
3. Authority to investigate pollution and to issue orders against polluters, requiring abatement of pollution.
4. Power to seek injunctions when necessary to protect the public interest.
5. Power to review plans of new waste systems and treatment works prior to construction and to require suitable treatment.
6. Control over the maintenance and operation of waste treatment works.
7. Power to order the construction of sewage and waste treatment works.

GENERAL PLAN OF STREAM POLLUTION CONTROL PROGRAM FOR THE STATE OF TENNESSEE

General Objectives

1. To unify and co-ordinate the efforts of the various public and private agencies concerned with the sources and effects of water pollution in order to reconcile conflicts of interest and maintain a consistent and equitable state policy toward pollution control.

2. To exercise general supervision and control over the use of surface waters for sewage and waste disposal so as to secure the maximum benefits to the state from these and all other essential uses of its water resources.

3. To maintain suitable sources of public water supplies so that, with a reasonable degree of purification, a safe and satisfactory water may be provided wherever necessary for domestic and other municipal use.

4. To protect the relatively unpolluted and, as far as possible, to improve the polluted waters in order to promote desirable fish and other aquatic life and to provide suitable places of wholesome recreation.

5. To conduct a reasonable but definite program, including technical cooperation and guidance, to obtain the

treatment of municipal sewage and industrial wastes as found necessary to prevent excessive water pollution.

6. Simultaneously, to foster industrial expansion and the development of natural resources by maintaining suitable waters for industrial use; and by co-operative studies to work out practicable and economical methods for the treatment and disposal of various industrial wastes in cases where satisfactory methods are not already known.

Essentials of State Legislation

A. Control Agencies and Authority

1. *The Stream Pollution Control Board.* This board, newly created to represent the major interests concerned, is given authority for supervision of all water pollution problems and activities in the state; and is charged with specific duties and responsibilities for water pollution control.

2. *Administrative Agency.* On behalf of the board and subject to its general policies and directives the State Department of Public Health, with its existing Division of Sanitary Engineering, is designated as the administrative agency to carry out the continuous and detailed program of the board. This includes: correspondence; sanitary engineering service; co-operative surveys and investigations; supervisory activities; and integration of the efforts of all state departments and of other agencies into the unified program of pollution control.

B. Facilities for the Control Program

1. *Personnel.* The board is authorized to employ technical and other personnel and to accept the part-time or full-time assignment of workers, available for pollution control activities, from state departments or elsewhere. This provides for field, laboratory and office work; expert consultation when needed; legal advice (from the State Attorney General) regarding any aspects of the program; and assistance in legal prosecutions, if necessary.

2. *Other Facilities.* The board is authorized to provide or to accept from other agencies necessary space and equipment for office or laboratory use, travel, sampling programs and other purposes as needed.

3. *Funds.* State appropriations for the regular program of pollution control are earmarked for this purpose. For special co-operative projects the board may receive contributions of funds, set up joint budgets upon which all contributors and the board agree and use such funds only as agreed.

Major Functions and Limitations of the Board

A. Functions

The more important general and specific functions of the board are:

1. To determine the present and probable future condition of all important streams and lakes of the state with respect to pollution; and the various uses of these waters, including sewage and industrial waste disposal, that are in the public interest.

a. Scientific field and laboratory surveys of particular streams and lakes.

b. Collection and analysis of data concerning the quantity and character of pollutional materials from existing or proposed waste disposal systems.

c. Studies of the present and probable future essential uses of these waters. This will require information concerning the needs for: domestic water supplies, fish and other aquatic life, recreational areas, industrial water supplies, agricultural uses, disposal of municipal sewage, disposal of industrial wastes, and other purposes that are related to pollution.

2. To define the maximum degrees of pollution that will be permitted under various conditions. This may include:

a. Adoption of pollution standards designating the minimum qualities of water considered acceptable in relation to particular water uses.

b. Adoption of general policies, for the state as a whole, for particular drainage basins or under specified conditions of water usage in order to define the attitude of the board toward pollution control. For example, such policies might state the time allowed for conformity with pollution standards adopted; priority of certain uses (as for public water supplies) over other uses; or minimum degrees of treatment to be provided on all streams in the area covered.

c. Classification of streams according to present qualities of water and desirable conditions to be attained in the future to serve as a guide for a gradual long-range program of improvement and control.

3. To exercise supervision over the discharge of sewage, industrial wastes and other wastes into streams and lakes in so far as they might affect the public. This may involve a number of actions and activities, including:

a. Adopt rules and regulations setting forth the procedures and conditions that must be observed in order that the disposal of sewage or wastes into surface waters may be approved.

b. Collect detailed information regarding individual municipal and industrial waste systems (existing or proposed). This may be done by field investigations or by requiring the submission of descriptive data, laboratory samples and waste treatment reports from the municipalities and industries concerned.

c. Determine and recommend to municipalities, industries or other owners any needed improvements in waste disposal systems or their operation; and, finally, if necessary, issue orders demanding that essential improvements be made within a reasonable time. Legitimate orders of the board, if ignored, can be enforced by court action.

d. Supervision of the installation and operation of systems for waste treatment and disposal. This should include review and approval of plans before construction is commenced; sanitary engineering consultation and field inspections; and periodic review of laboratory and operations records to assure that satisfactory disposal is maintained.

4. Research studies conducted by the board or jointly by voluntary agreements with other agencies either to improve survey procedures or, more especially, to find more effective or practicable methods of treatment and disposal for sewage and industrial wastes.

a. Joint studies are advantageous, for example, where pollution is serious from industrial wastes or municipal sewage mixed with industrial wastes and knowledge of effective treatment methods is not sufficient to justify specific recommendations by the board.

b. The allotment of funds for joint studies should follow the principle that the state is responsible for the costs of assuring protection of the public while the municipalities and industries are responsible for the costs of solving their problems of waste disposal.

5. To deal with interstate problems of pollution with which the state of Tennessee is concerned because of pollution in waters either entering or leaving the state.

a. Co-operate with water pollution control agencies of neighboring states and with federal agencies to effectuate consistent pollution control measures for entire drainage basins irrespective of state lines.

b. To represent and act on behalf of the state in the development and execution of interstate agreements and compacts for the joint control of interstate pollution by two or more states.

c. Finally, if all other means fail and the urgency of such interstate problems justifies more forceful action, to recommend and assist in preparing for and carrying out appropriate legal action to obtain necessary protection or improvement of interstate waters.

6. To co-operate especially with state and local governmental agencies that are responsible for waste disposal. This may include technical advise and consultation, support for necessary administrative or financial arrangements for sewage treatment, or other feasible assistance. Examples of such agencies are:

a. Cities and towns

b. Sanitary districts

c. State and county hospitals or other institutions

7. To carry on a constructive program of education and releases of information concerning water pollution problems and the practical aspects of pollution control. The general purpose of educational activities would include:

a. To inform the public of the value of surface waters, the effects of pollution, the possibilities, limitations and costs of pollution abatement, and progress being made in stream protection and improvement.

b. To induce municipalities, industries and other owners of waste systems to accept their responsibilities for proper disposal to prevent harm to the public and to provide necessary waste treatment and disposal facilities.

c. To provide technical training and information for sewage and waste treatment plant operators or others responsible for the continuous maintenance and operation of waste disposal systems.

B. Limitations to Assure Reasonable Action by the Board

Because of the complex and variable aspects of water pollution problems, broad powers to be exercised with judgment and discretion are necessary to enable the board and its administrative officials to function fairly and effectively. In addition to the usual and existing legal safeguards against arbitrary and unreasonable action by state

agencies the suggested bill creating the board provides that:

1. Jurisdiction of the board is limited to:

   a. Control of pollution of such a character and degree that it may harm the public or interfere with reasonable and necessary uses of surface waters, and

   b. The establishment of pollution standards that are in the interest of the public.

   Note: All other authority of the board is granted and can be legally used only as a means of accomplishing "1-a" and "1-b" above.

2. In the course of its investigations the board cannot require the disclosure of secret formulas or what might justifiably be termed "trade secrets."

3. Special safeguards against summary, unnecessary or unreasonable orders by the board are specifically provided. These include:

   a. True and accurate records of action by the board kept in the office of the Technical Secretary.

   b. Filing of general standards, regulations and orders with the Secretary of State at least 30 days before they take effect.

   c. Prompt hearing of any person in interest who has a grievance or objection against any action of the board.

   d. Reassurance of the right of aggrieved persons to appeal to the proper courts for final decision upon the necessity and legality of any action by the board.

Administrative Organization and Procedures

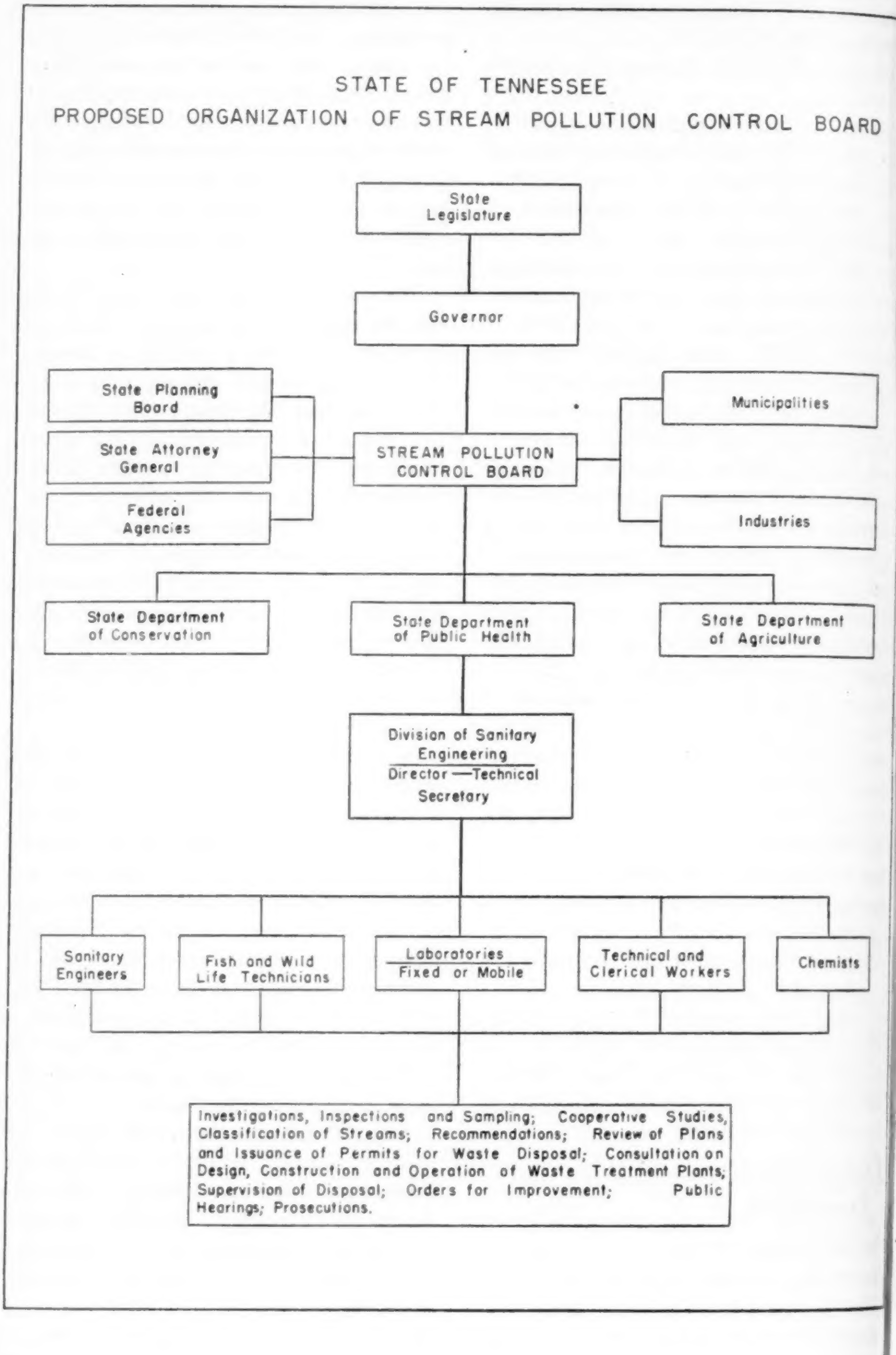
A. Organization

Although water pollution control must be based upon state law, constructive results from this program will depend principally upon its organization and administration. The principles of a unified program have been discussed in the preceeding report and incorporated in the legislative act (now before the Tennessee Legislature). The general scheme of this organization, as proposed for Tennessee, is shown in the organization chart below.

The functions outlined previously and the units of organization shown in the chart conform generally with successful programs of other states, notably Wisconsin, Michigan and Illinois. The center of authority is the Stream Pollution Control Board. The board represents all major interests concerned with water pollution and is responsible for all general policies, regulations and general supervision of the entire program. The board also must give full and fair hearings of complaints or objections and authorize enforcement proceedings if they are found necessary.

Investigations, recommendations and state service constitute the bulk of the program. Being concerned chiefly with the needs and methods for waste treatment these are largely sanitary engineering functions and, therefore, are centered in the Division of Sanitary Engineering of the State Department of Public Health. A close tie-in of the policies of the board with their execution is provided by having the Director of Sanitary Engineering serve as technical secretary of the board.

The actual number and types of workers on the technical staff would depend upon the problems encountered, the facilities available and the decision of the board to concentrate upon one or another aspect of pollution at any particular time. Also it would depend upon the interest of other groups or


STATE OF TENNESSEE
PROPOSED ORGANIZATION OF STREAM POLLUTION CONTROL BOARD
State Legislature
Governor
State Planning Board
State Attorney General
Federal Agencies
STREAM POLLUTION CONTROL BOARD
Municipalities
Industries
State Department of Conservation
State Department of Public Health
State Department of Agriculture
Division of Sanitary Engineering
Director—Technical Secretary
Sanitary Engineers
Fish and Wild Life Technicians
Laboratories
Fixed or Mobile
Technical and Clerical Workers
Chemists
Investigations, Inspections and Sampling; Cooperative Studies, Classification of Streams; Recommendations; Review of Plans and Issuance of Permits for Waste Disposal; Consultation on Design, Construction and Operation of Waste Treatment Plants; Supervision of Disposal; Orders for Improvement; Public Hearings; Prosecutions.

agencies in particular activities and their willingness to contribute funds or personnel to strengthen them. For example, the Department of Health might temporarily assign a water supply engineer and the Department of Conservation a fish expert to assist in determining more fully the effects of pollution or the results of waste treatment in a particular stream or area; or an industry might assign a specialized chemist to work with the board's sanitary engineers in an effort to improve its waste treatment facilities. The authority of the board and the organization must be flexible in order to provide an efficient technical service.

The details of interdepartmental relationships are not shown on the organization chart, but are covered in the proposed legislative act. Unity in the pollution control program without sacrifice of autonomy or authority by individual state departments is the guiding principle. For example, if one of the state departments should assign a worker to this program, it would not lose control of the worker except as agreed voluntarily; but any work done by him while thus assigned must be under the general supervision and technical direction of the board.

B. Procedures

The exact nature and sequence of action necessary to meet the various types of problems must be worked out to fit the particular circumstances and the development of the entire control program. As guides in adopting procedures, the board would have available reports of activities by pollution control boards or commissions in other states; and the long-established program of the Division of Sanitary Engineering in the investigation, improvement and supervision of water works, milk supplies and sewerage systems for the control of health hazards.

Some examples of specific actions which this program might include are outlined below. They exemplify a logical and orderly approach to two typical problems: (1) to obtain improvement in the conditions of pollution in an entire drainage basin; and (2) to decrease the pollution from a large industry or group of industries for which satisfactory disposal methods are not known.

1. *Improvement in Entire Drainage Basin*

Directly or indirectly, the board might take the following steps:

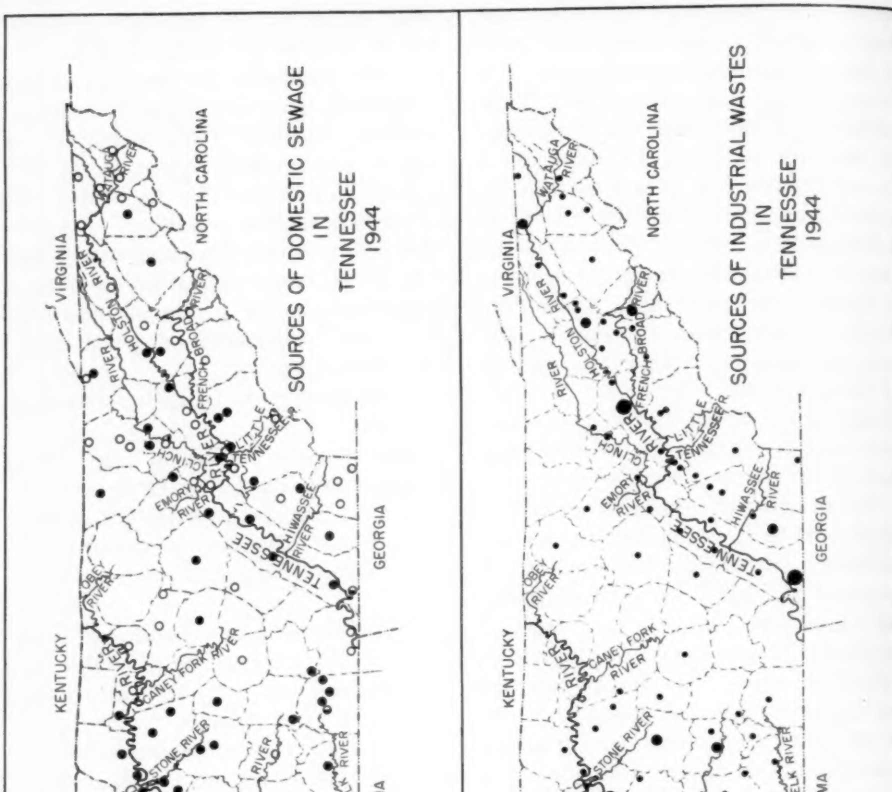
a. Bring survey data up to date
   (1) Condition of streams
   (2) Sources of wastes
   (3) Uses of water

b. Decide what minimum qualities of water it is desirable and feasible to maintain in various sections of streams.

c. Determine minimum degree of treatment (if any) that is necessary for each source of sewage and industrial waste.

d. Informal conferences with municipal and industrial officials concerning the feasibility of providing necessary treatment and the amount of time required. The purpose is:
   (1) To explain the basin-wide proposal.
   (2) To obtain practical knowledge of any special factors or difficulties.
   (3) To solicit co-operation in a concerted effort toward improvement.

e. Formal adoption of minimum pollution standards for various sections of the streams in the basin.


SOURCES OF DOMESTIC SEWAGE
IN
TENNESSEE
1944
SYMBOLS
SEWERAGE SYSTEMS
NONE OF SEWAGE TREATED
PART OF SEWAGE TREATED
ALL OF SEWAGE TREATED
SOURCES OF INDUSTRIAL WASTES
IN
TENNESSEE
1944
SYMBOLS
NUMBER OF INDUSTRIES
1-3
4-7
8 OR MORE
KENTUCKY
VIRGINIA
NORTH CAROLINA
GEORGIA
ALABAMA
MISSISSIPPI
ARKANSAS
MISSOURI

f. Recommendations to individual owners of existing waste systems.
g. Public hearings, if any are demanded, followed by final determination of improvements to be required.
h. Engineering consultation and field visits to assist in planning disposal improvements.
i. Review of applications, plans, specifications and engineering data; and issuance of permits for approved treatment and disposal systems.
j. Issue of special orders to any owners who have not shown effort and reasonable progress toward waste treatment as recommended.
k. Follow up of special orders—consultation during and further action at end of time limits specified in each order.
l. Review of plans and issuance of permits for proposed new waste discharges with treatment to conform with the pollution standards previously adopted.
m. Supervision of operation of sewage and waste treatment plants to assure the continued protection for which they were designed.

2. Disposal of Industrial Wastes for Which Satisfactory Methods Are Not Known

In this situation the board could not act with intelligence and assurance in making recommendations (1-f above). The most practical procedure is for the board to join with an industry or a group of similar industries and make studies to find methods of disposal which would meet the pollution standards adopted without prohibitive cost to the industry.* As soon as such methods were worked out the normal procedure, outlined above, would be followed.

Reports of numerous joint studies in other states indicate that such a study project might well include the following steps:

a. Organize a technical committee to plan and conduct the studies.
b. Board and industry agree on general features, provide funds and adopt a general plan for the studies.
c. Technical committee obtain facilities and conduct experimental work. Most of this is done at the industrial plant under operating conditions and considers all possibilities of:
(1) Waste recovery and utilization in the regular industrial processes or as byproducts.
(2) Segregation and separate disposal of strong wastes excluding them from the stream if possible.
(3) Various processes of waste treatment.
d. Analysis of data and report of results from the experimental studies.
e. General design and layout of equipment to apply the disposal methods recommended by the technical committee.

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* Various industry groups already have provided engineering and technical advisers to assist in joint action to solve their common waste disposal problems. Examples are the American Pulp and Paper Association and the American Petroleum Institute.

Experiences With Mechanical Joint Pipe

By E. R. Sharp

Chief, Bureau of Gas and Water Distribution, Richmond, Va.

Presented on Nov. 15, 1944, at the Virginia Section Meeting, Richmond, Va.

UNTIL the spring of 1942 the Richmond Bureau of Gas and Water Distribution had used very little mechanical joint pipe. At that time, however, a gas line, approximately 2.6 mi. long, for high-pressure gas, was installed for a badly needed supplementary line in South Richmond.

The Cast Iron Pipe Research Association, realizing the confusion created by the great variety of patented mechanical joint pipe, is introducing a standardized procedure (Figs. 1–6) which was followed in Richmond. The successful results on this line led Richmond to make a further study of the use of mechanical joint pipe in laying mains.

Main Installation at South Richmond

After installing the main with a Dresser coupling joint, the pipe and joints were tested with air, in two sections, by pumping the pressure up to 50 lb. A recording gage was installed and allowed to remain for 24 hours and a chart of each test was obtained. While testing, two or three joints were found to be leaking air but when the bolts were tightened these leaks disappeared. After this correction was made no pressure loss occurred. As a matter of fact, the pressure varied according to the temperature of the air—decreasing during the night and increasing with the heat of the day.

Mechanical joint pipe can be jointed on the ground by the side of the trench and then, if the equipment for lowering is available, lowered into the trench. This, of course, can be done only in new sections where other underground utilities will not be encountered. Where gas and water services and other utilities exist it is obviously impractical to lay the pipe on the bank and lower it into the trench. Although this has been done successfully on several occasions, it has been found simpler to place blocks in the trench, lay the pipe on them and later lower the assembled pipe to its final position.

The Richmond Bureau of Gas and Water Distribution lays gas and water mains side by side in the same trench. The flexibility of the joints permits the pipe to move up and down and sideways, without disturbing any of the joints. As a rule the gas pipe is assembled on the blocks first. After a slight hole is dug for the flange and bell to permit the body of the pipe to rest on the grade of the trench, the blocks are removed. The same procedure is then followed for the water pipe, which is lowered after assembly on the bank into the trench. There is no trouble in removing the blocks and lowering 3- and 4-in. pipe to position by hand, but on 6-in. and larger pipe the men have difficulty in lifting the pipe to remove the blocks. To over-

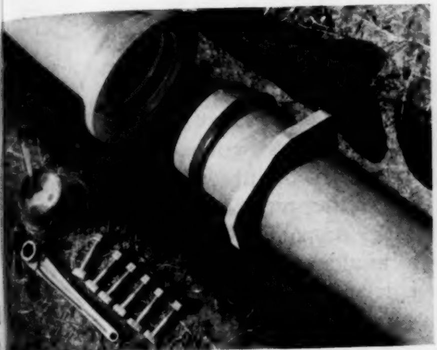

FIG. 1. Rubber Gasket and Follower on Spigot End of Pipe

FIG. 2. Spigot End of Pipe Entering Into Bell

FIG. 3. Soapsuds Applied to Rubber Gasket

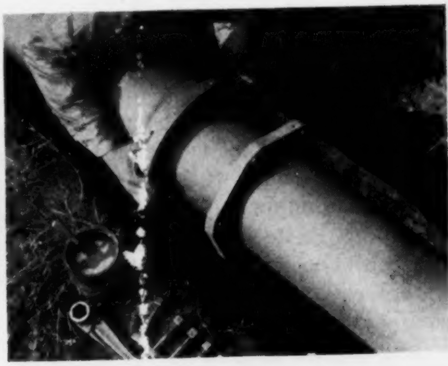

FIG. 4. Rubber Gasket Forced Home in Socket With Fingers

FIG. 5. Bolt and Nuts Inserted in Flange and Follower Ring

FIG. 6. Tightening Up Nuts With Rachet Torque Socket Wrench

come this difficulty a device suggested by one of the men and consisting of a wire puller to raise and lower the pipe, with a timber across the top of the trench and a loop around the pipe, has proved very successful.

Each year, with the beginning of warm or cold weather, Richmond has experienced a large number of water main breaks. The temperature of the James River, from which Richmond derives its supply, varies approximately 50 deg., causing the mains to expand and contract. In most of the breaks the pipe has been broken completely around, as though a pipe cutter had been used. It is hoped to eliminate this trouble by the use of flexible mechanical joint pipe.

Richmond is using for water pipes the U.S. Mechanical Joint-type, 250-lb. class, cement-lined, emulsion-treated, cast-iron pipe in 18-ft. lengths. Two-thirds the number of joints are required in 18-ft. lengths as in 12-ft. lengths. The joint is flexible and can be laid with a deflection of 3°–30′ in each joint, amounting to 8.8″ in 12-ft. lengths and 13.2″ in 18-ft. lengths. When using 12-ft. lengths the pipe can be laid on a curve with a radius of 196 ft. and when using 18-ft. lengths, 295 ft.

No particular skill is needed to lay the pipe. Anyone with an open-end wrench can bolt the pipe together. There is no calking to be done nor is it necessary to pour any joints. An open-end wrench can be used for bolting the pipe together but a ratchet torque socket wrench is used for tightening the bolts. A 50-lb. pull on a 10-in. wrench is recommended but laboratory tests indicate that a 25-lb. pull on a 10-in. wrench is ample to hold the working pressure of the pipe. Richmond sets the wrench to break pull at 60 lb. When a 50-lb. pull was used several small leaks occurred, but since using 60 lb. no trouble has been encountered.

The flexibility of mechanical joint pipe is of particular advantage when laying in muddy and wet trenches. Since the pipe is laid on blocks, it makes no particular difference how wet or muddy the trench is. The joint is high and dry and is wiped free of dirt or mud and painted with soapy water. The rubber ring is then pulled into the joint by the following ring, the bolts are tightened and the joint is complete. After the joint is bolted up, the blocks removed and the pipe in place, the joint is not affected by the mud or water regardless of how muddy the trench may be. Because of its flexibility, no effort is necessary to keep the pipe in position.

After the line is completed the pipe can be re-aligned from one end to the other. In laying the pipe a gap of 0.01 ft. is allowed in each joint to prevent expansion, which would cause the pipe to buckle. This is not necessary if the pipe is laid in warm weather, but because of the depth of the bell it is a safe precaution and a good habit to form.

It is a well-known fact that jute packing and braided hemp have often been sources of contamination. On several occasions before using the mechanical joint, considerable difficulty was encountered in sterilizing the mains. Often it was necessary to use a portable chlorinator for quite a period before receiving a negative report from the chemist. Since using mechanical joint pipe, wherever a residual of chlorine could be obtained, successful results have been obtained after flushing through a 1-in. service for a period of three or four days. Although it has

not yet been proved, it is believed that rubber rings have caused less trouble than jute or hemp.

Since laying water mains with the mechanical joint, only one leak has occurred during the test before backfilling. In this case the joint was unbolted and the rubber ring pulled out but there was no apparent defect. After reassembly the joint did not leak so it was assumed that a small piece of gravel or other debris had caused the leak.

Standard fittings in mechanical joint pipe are all bell. Due to Richmond's policy of installing a valve on the fitting, this has caused some inconvenience. It is hoped that the manufacturers of valves and fire hydrants will co-operate to make obtainable a valve with a spigot on one end and a mechanical joint on the other; and that they will also make a mechanical joint on the hub of a fire hydrant. This will eliminate the necessity of using a short nipple between the fitting and valve.

Erratum

McLaughlin, Martin J. Water Conservation in Philadelphia. Jour. A.W.W.A., **37**: 66 (1945).

In "Water Conservation in Philadelphia," by Martin J. McLaughlin, in the January Journal, last paragraph, first column, page 66, reference is made to the quantities of water consumed on the maximum day, the average of the maximum days for the first nine months of 1944, and the average daily consumption during the first two-thirds of the year 1944. The figures are given as "gallons." The figures should have been given as "million gallons per day." The passage should read:

> The maximum day for 1944 occurred on July 12 when 387.4 mgd. were consumed. The average of the maximum days for the first nine months was 353.1 mgd. The average daily consumption for two-thirds of the year was 328.4 mgd.

Water Conservation in Rahway, N. J.

By Mortimer M. Gibbons

Supt., Board of Water Commissioners, Rahway, N.J.

Presented on Nov. 4, 1944, at the New Jersey Section Meeting, Atlantic City, N.J.

RAHWAY, N.J., is located at the lower end of the Rahway River Basin, at which point there is a watershed area of approximately 40 sq.mi. Five different water supplies, located above Rahway, divert an average of 10 mgd. surface and ground water from this basin. No appreciable raw water storage is available at Rahway. The shallow pondage above a low diversion dam has an available capacity of about 1 mil.gal. The city is thus dependent for water supply upon a continuous flow in the river. The dry weather river flow in 1944 was 4.0 mgd. The consumption for the year ending Sept. 30, 1943, averaged 4.67 mgd. On the peak day of this year, the consumption was 6.31 mgd.; the peak hourly rate was 7.8 mgd.

Water consumption in this industrial area has soared in recent years. Three of the five water supplies above Rahway have applied for additional diversion rights in the past four years. The New Jersey State Water Policy Commission has wisely followed a conservative policy in apportioning the water of this basin. Rahway has likewise been hard-pressed to satisfy the mounting industrial demand. In 1944, industrial consumption comprised about 63 per cent of the water pumped and 75 per cent of water sales. Three Rahway interconnections with two neighboring water supplies were constructed, with the co-operation of the New Jersey State Water Policy Commission. Water is purchased during peak loads or an extended drought. However, only one of the interconnections is a firm supply under drought conditions.

Under these circumstances, the need for water conservation in Rahway was clearly apparent. The industrial water consumers, as well as part of the commercial consumers, were on a metered basis. All domestic water service was on a flat rate basis. Excluding metered industrial consumption, the remaining water consumption averaged 104 gpd. per capita, as estimated from the master meter readings. This indicated that domestic and intermediate consumption was rather high but not excessive. Relatively low water main pressure of 45 psi. and the existence of many old corroded house connections probably kept the flat rate consumption at this level. The need for saving water, however, was so great that the water board decided to install domestic meters in 1941 at its own expense.

Meter Installation, Reading and Maintenance

Over 4,200 domestic meters and meter connections were purchased in 1941 and 1942. These were installed by local plumbers under contract in 1942 and 1943. Three classes of in-

stallation with different payment rates were set up: (1) basement settings in which meter couplings or copperhorns could readily be inserted in consumer piping; (2) basement settings requiring changes in consumer piping; and (3) curb settings.

A tap number was assigned to each lot in the city on a set of tax maps. Each meter installed was identified on a meter reading sheet and office card by its appropriate tap number. The city was divided into three routes, each with four sections. Each section was divided among three meter reading books. The average number of consumers per book was about 120.

Each route is read and billed once every quarter year. Reading and billing are continuous throughout the year, thus spreading out the work. The date of mailing is stamped on the bill. A period of fifteen days thereafter is allowed for payment before a penalty of five cents per month for unpaid bills is imposed. When a reading is missed, a business reply card, with a facsimile of the meter dials, is left at the house for consumer marking. Industrial meters are read monthly but billed quarterly in a separate group.

A meter repair shop and testing laboratory was set up for meter maintenance. When governmental restrictions prohibited the sale of bronze-cased meters, a sufficient number of old meters were rebuilt with new parts to complete the metering program. Meters are maintained at the expense of the water utility except for damage due to freezing or hot water.

Consumption After Domestic Metering

The effect of metering on domestic and intermediate water consumption is shown in Table 1. The results of the first full year after metering are compared with the average of the four preceding years. During this five-year period, general business conditions were uniformly good and the population was relatively stable. The consumption figures are master meter readings less metered industrial consumption.

TABLE 1

Effect of Domestic Metering

Period	Domestic and Intermediate Consumption, *mgd.*		Saving *mgd.*
	Before Metering 4-Year Average	After Metering 1 Year	
1st Quarter	1.83	1.61	0.22
2nd Quarter	1.90	1.51	0.39
3rd Quarter	1.96	1.61	0.36
4th Quarter	1.83	1.63	0.20
Year	1.88	1.59	0.29
Per Capita Consumption, *gpd.*	104	88	16 (15.4%)

It may be seen that the average annual water saving after domestic metering was 290,000 gpd. or 16 gpd. per capita. The saving in the second and third quarters exceeded the yearly average, being 22 and 20 gpd. per capita, respectively. The average annual per capita consumption declined from 104 to 88 gpd. This represented a saving of approximately 15 per cent.

Table 2 shows a comparison of domestic water consumption for two years, the year preceding domestic metering with the year following metering. This was done to eliminate any possible effect of abnormal water loss by

TABLE 2

Comparison of Two Years' Results

Period	Before Domestic Metering, *mgd.*					After Domestic Metering, *mgd.*				
	Consumption			Daily Peak Hour		Consumption			Daily Peak Hour	
	Total	Industrial	Domestic	Average	Maximum	Total	Industrial	Domestic	Average	Maximum
1st Quarter	4.45	2.69	1.76	5.9	6.9	4.32	2.71	1.61	5.5	6.7
2nd Quarter	5.16	3.09	2.07	6.2	7.8	4.03	2.52	1.51	5.5	6.4
3rd Quarter	4.88	3.10	1.78	6.2	7.1	4.34	2.73	1.61	5.7	7.1
4th Quarter	4.20	2.32	1.88	5.5	7.0	4.40	2.77	1.63	5.6	6.9
Year	4.67	2.80	1.87	5.9	7.8	4.27	2.68	1.59	5.6	7.1
Per Capita Consumption, *gpd.*	259	155	104			237	149	88		

leakage when the five-year period was considered. A water leak survey was completed just prior to the two-year period.

It may be seen that the comparison is about the same whether the two-year or five-year period is used. In each case, metering reduced the annual average domestic and intermediate water consumption from 104 to 88 gpd. per capita, a saving of 16 gpd. per capita. The greatest saving was in the second quarter of the two-year period when a reduction of 31 gpd. per capita was realized.

Peak Loads

In addition to the quantity of water saved, domestic metering has had a beneficial effect in reducing daily peak-hour loads. This is important in a plant with a rated filter capacity of 6 mgd., small pure-water storage and hourly peaks above 7.0 mgd. Since industrial consumption varies from quarter to quarter, the average peak hours shown in Table 2 are not always comparable. However, in the first quarter, the industrial consumption was practically identical in both years. It may be seen that after domestic metering, the average daily peak was 5.5 mgd. in the first quarter as compared with an average peak of 5.9 mgd. in the same quarter during the preceding year. This reduction of 0.4 mgd. in average peak hourly load seems properly attributable to domestic metering.

Analysis of Water Consumption

Metered consumption records for all classes of water service have enabled us to break down the total consumption into its components, as shown in Table 3. These figures are for the first year after 100 per cent metering and after a water leak survey of the distribution system. The total consumption is 237 gpd. per capita of which 207 gpd. per capita is metered. The remaining 30 gpd. per capita, or 13 per cent of the water pumped, is unaccounted for. It is believed that part of this appreciable loss may be due to seepage from the considerable mileage of old concrete mains in the distribution system.

TABLE 3

Analysis of Per Capita Consumption (After 100% Metering)

Consumers	Consumption gpd. per capita	Percentage
Industrial	149	63
Intermediate	7	3
Domestic	46	19
Public and Free	5	2
Unaccounted	30	13
Total.........	237	100

These old concrete mains were installed from 40 to 70 years ago. They consist of a "stovepipe" of planished iron sandwiched between an inner and an outer layer of Rosendale cement. The iron seam is riveted along its entire length. Occasionally, when a concrete main is uncovered before rupture, it is found leaking along the riveted seam. This class of pipe is now being replaced with cast iron.

Economics of Domestic Meter Installation

The domestic meters saved 16 gpd. per capita, or 106 mil.gal. per year. They reduced peak-hour loads, provided valuable information as to the extent of water loss in the distribution system and established an equitable basis for charging for water service. But were they worth the cost? In Table 4, the cost data and yearly operating expense and saving are summarized.

The over-all meter installation cost \$14.976 per unit in place. The basement settings cost \$13.969 per unit and the curb settings \$28,495 per unit. The total cost was \$65,895.16. This money cost 2 per cent per year for interest. Depreciation is figured on a 30-year basis at 2 per cent compound interest. The increased operating charge totals \$7,000 per year. This provides one meter reader, one meter repairman and setter, one clerk in the office and meter repair parts and transportation.

The savings consist of a reduction in the state diversion charge,* reduction in pumping and purification costs and an estimate of the fair value of the water saved. The latter item is the crux of the savings estimate. It was appraised on the following basis:

Water is purchased during peak loads and drought periods. The margin between available river flow and city consumption is very low during part of the year. It was assumed that a safe excess river flow should be equal to the maximum day's consumption to provide for emergency demands and sanitation. There were six months in

* The state diversion charge is a "tax" placed upon the diversion of surface waters for public and potable use by municipalities and private water companies diverting water from streams, reservoirs and lakes within the state. This charge is made in accordance with the provisions of Revised Statutes 58, Chapter 2. The charge is determined by the amount of water which the divertor uses during a calendar year in excess of a so-called "free allowance" which, in turn, is determined either by the amount of water which the divertor used in the year 1907 (when the law was passed) or an amount based upon the census of 1905 of the territory supplied at 100 gal. per capita, the larger amount being used. The rate charged for the excess water diverted varies from a minimum of \$1.00 per mil.gal. to a maximum of \$10.00 per mil.gal., depending upon the depletion of the stream flow below the point of diversion as compared with the average flow of the stream for the driest month of record or, in lieu of stream flow records, at a rate of 125,000 gpd. for each square mile of unappropriated watershed above the point of diversion. If a flow equal to the average daily flow for the driest month of record, or the 125,000 gpd. rate is maintained below the point of diversion, a minimum charge of \$1.00 per mil.gal. is applied. If the flow at any time is less than said amount then the rate is increased up to a maximum of \$10.00 per mil.gal. in case no flow is maintained below the point of diversion. The exact charge in between these two extremes is determined by a formula which has been adopted by the Commission. *H. T. Critchlow, Chief Engr., New Jersey State Water Policy Com.*

TABLE 4

Economics of Meter Installation

		Amount
Cost		
Material—4400 meters and meter connections, including 305 curb settings @ $11.122		$48,936.63
Installation—4095 inside settings @ $2.847		11,659.83
305 curb settings @ $17.373		5,298.70
TOTAL COST—4400 @ $14.976		$65,895.16
Additional Annual Expense Over Flat Rate System		
Cost of repairing, setting, reading and billing	$ 7,000.00	
Depreciation—30-yr. basis @ 2% compound interest	1,621.01	
Return on investment (2% of $65,895.16)	1,317.90	
TOTAL ANNUAL EXPENSE		$ 9,938.91
Annual Savings Due to Domestic Meters		
Water Utility		
Reduction in state diversion charge—106 mil.gal. @ $2.00	$ 212.00	
Reduction in pumping and purification costs—106 mil.gal. @ $20.00	2,120.00	
Fair value of firm supply of water saved, ½ x 106 mil.gal. @ ($167 − 20 = $147)	7,791.00	
TOTAL ANNUAL SAVINGS		$10,123.00
City		
Reduction in sewage disposal costs (estimate)		$ 1,000.00
Net Annual Savings		
Water Utility—$10,123.00 − $9,938.91	184.09	
City	1,000.00	
COMBINED SAVING		$ 1,184.09

the year, ending Sept. 30, 1944, in which the median excess river flow was less than the maximum day's consumption. Consequently, one half of the total water saved by metering was credited as a saving in a critical period. This portion of water was valued at the actual cost of purchased water, $167 per mil.gal. minus $20 per mil.gal. for pumping and purification costs.

On this basis, the total annual saving due to domestic meters slightly exceeds the total annual expense, leaving a net saving of $184.09 per year. In addition, it is estimated that the city will save $1,000 per year in a reduction of sewage disposal costs. In short, the investment in domestic meters appears to be a sound one.

Discussion

Harry E. Jordan

Mr. Gibbons' account of the city-wide installation of water meters at Rahway as a conservation measure during wartime is significant and merits study by every water department executive who faces increased demands for service at a time when major improvements and extensions cannot easily be undertaken.

When this project started all domestic customers were on a flat rate basis. When the job was done, every customer was metered. That in itself was quite a public relations job and for the success of it, Mr. Gibbons and his associates, as well as the city administration of Rahway, which must have stood behind the task, deserve praise.

Analysis of per capita domestic demand before and after metering discloses that a reduction from 104 to 88 gpd., or 15 per cent, was effected. This totaled approximately 300,000 gpd. and in a community dependent for its supply upon the continuous flow of a river, the dry weather flow of which was less than the average daily demand of the city, a third of a million gallons becomes a significant amount.

The recorded domestic consumption before metering was not one of those glaringly excessive amounts of which examples can be found across the country. But even after metering, it does not appear to be low.

Pond's analysis of urban domestic water consumption [1] contains a series of valuable figures and comments. It is pointed out, for example, that the per capita daily demand for water in single-family houses varies with the economic status of the occupants. The range in a group of several hundred houses in New Haven, Conn., was from 34 gpd. per capita for families of "fair" economic status to 85 gpd. per capita for one group termed "excellent." Another group of 290 houses in the "excellent" status averaged 125 gpd. per capita, probably the two or three bathroom, two car, large lawn and flower border type of dwelling.

Cutting across the various levels of single-family status, Pond indicates that on an average in all single-family houses, domestic consumption need not be over 50 gpd. per capita. Multiple-family houses use less water—about 40 gpd. Apartment dwellers, on the other hand, waste water, especially hot water, and consume on the average 75 gpd. per capita. Large-scale housing projects need not consume more than 45 gpd. per capita.

Thus we may deduce that while the effect of metering domestic consumers in Rahway has been fully creditable, it has not reached the minimum that is practicable. The further reduction that can be attained has value not alone as a wartime conservation measure, but can actually reflect itself in the long-term planning of replacements or additions to the city's water works system. This is a type of saving which the author would find difficult to estimate and which has not been set up in Table 4.

Another important item in Table 4 is the estimate of "cost of repairing, setting, reading and billing." For the 4,400 meters installed, the author has estimated an annual expense of $7,000. This is slightly less than $1.60 per

[1] Pond, M. A. Urban Domestic Water Consumption. Jour. A.W.W.A., **31**: 2003 (1939).

meter per year, with domestic meters read on a quarterly interval. The estimate appears to be quite, if not overly, adequate. In one large water utility in the Middle West, with a *monthly* reading schedule, the annual cost of reading, billing, servicing and maintenance is $0.93 per meter per year. With quarterly reading, it is estimated that this plant could reduce the charge to $0.55 per meter per year. Naturally such costs vary inversely with the number of meters in service. But on a quarterly reading basis, it would appear that from $0.75 to $1.00 per meter would be an adequate allowance. This would reduce the author's item of $7,000 for reading, billing and maintenance of 4,400 meters to $4,400 on the $1.00 basis and $3,300 on the $0.75 basis and would increase the net annual savings to the water department by an amount ranging from $2,600 to $3,700. The difference is very significant for a city the size of Rahway. It is therefore suggested that this aspect of the author's estimates of savings be closely studied by persons who might at first glance be inclined to feel that the dollar saving derived from the installation of meters at Rahway was not great.

Mr. Gibbons' paper is, we must remember, directed toward conservation. The evidence contained in it shows that conservation at a critical period was effected. It is the writer's considered opinion that the domestic per capita consumption of water is still more than adequate for cleanliness and health and that a per capita domestic use of not more than 60 gpd. per capita could be attained by the customers of Rahway without any disservice to the community. More important, however, is the fact that the city has made the transition from the uneconomic flat rate basis for domestic water service to the all-metered basis where each customer pays for what he uses as well as for what he may waste. When a water department is operating on the fully metered basis it is in a position if its supply and facilities are adequate to promote wider use of water rather than to restrict it. The water works field, even in peacetime, is too greatly habituated to restricting the use of water. It should study the electric utility field and observe that it once operated on the flat rate basis, sold current principally for lighting and was a struggling industry. There came into that field men who sold the ideas of service and more service, of plant extension and readiness to serve every customer's wants, but on a metered basis. No country in the world has so great an electric service industry. No country uses so much current and no country has so great a number of conveniences related to daily living as has North America.

No continent has more adequate water supply services than North America. Nowhere else in the world is public water supply so adequate and so satisfactory in quality. But the public water supply industry on this continent generally yet has before it the problem of setting its sights to sell more water rather than less. Paradoxically, to reset those sights a water utility must first do just what has been done in Rahway—put all of its customers on a pay-for-what-you-get basis and view its operations from an allover economic basis as Mr. Gibbons has done. Having done this, and having set its productive house in order, it can then proceed to educate its customers to more useful consumption of water that will contribute to the health and general comfort of the family and the community.

The Meter Department of a Water Utility

A Symposium

By Seth Burnley, W. B. Harman and A. T. Lundberg

Presented on Nov. 14, 1944, at the Virginia Section Meeting, Richmond, Va.

INSTALLATION OF SERVICES AND SELECTION OF METERS—*Seth Burnley*

City Mgr., Charlottesville, Va.

DOMESTIC services for individual houses constitute at least 90 per cent of all services. Three-quarter-inch or 1-in. lines are usually sufficient for such installations. The main is simply tapped in the street, a corporation cock inserted in the line and the pipe, made of copper, lead, steel or iron, usually runs to the curb of the street where a curb stop and a meter are installed. Copper pipe had been very popular just prior to the war due to its flexibility, which eliminated fittings, compression or soldering joints. Pipe which is embedded in cinders should be protected by a small amount of sand around it. The meter is installed in a curb box and the line stubbed out to the property line from which point the property owner continues it into his house. The meter usually selected is either of a $\frac{5}{8}$-in. or a $\frac{3}{4}$-in. disc or propulsion type. All-bronze meters are preferable and are obtainable now, although for a year or so cast-iron bodies were the only ones available. Plastic cases will no doubt solve some of the reading difficulties.

There are, of course, a number of ways of setting the meter. Some prefer the rigid yoke or built-up cast-in-one-piece type and others use regular pipe connections. The main point, of course, is to install it so that it may be taken out easily for repairs or removal. There are numerous types, shapes and sizes of meter boxes, but cast-iron boxes with lids to fit are probably the most common. Concrete and vitrified clay is used extensively in some localities. The question of cover varies considerably. In cold climates a double lid with an air space affords good protection against freezing. One important point to remember is that many of these boxes are installed in the sidewalks and pedestrians pass over them continually. An insecure lid may result in a law suit.

The size of large meters can best be selected by finding the maximum requirements of the consumer and taking the rating of the manufacture. It is usually better to use a slightly smaller meter than the size of the pipe supplying it, as generally a more accurate measurement is secured if the meter runs somewhere near its capacity.

One type of large service which is often unmetered is that for automatic sprinkler systems in buildings. A service charge for installation may be, and in the author's opinion should be, made; however, there is probably more loss of revenue from meters in water works than in any other utility.

METER READING AND COLLECTION OF WATER BILLS—*W. B. Harman*

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About 95 per cent of the revenue of a water department is derived from the sale of metered water. The meter, therefore, is a very important part of the equipment for customer accounting. Its use has demonstrated beyond any doubt that it is the most practical, fair and efficient method of selling water. In other words, the meter provides a fail deal between producer and consumers.

It is important that all meters be kept in the best condition so that they will register accurately. This is the only means the water department has of knowing how much water has been delivered and how much the customer should pay. Unregistered water is a dead loss to the department and comes out of net revenue. Unless the meter registers correctly, is read correctly, the bill made out correctly and collected, the department cannot operate at a profit, nor can any reduction in water rates be made to the citizens who really own the water works nor, if the company is privately owned, can dividends be paid to the stockholders.

Reading Meters

Meters can be divided into two groups: first, the small meters, consisting of the $\frac{5}{8}$-, $\frac{3}{4}$- and 1-inch sizes, which are used on domestic services; and second, the large meters, which consist of all meters over the 1-in. size which are used on commercial and industrial services.

The small meters for domestic services can be read and billed quarterly, as the majority of domestic consumers do not use over their quarterly allowance of water. This does not mean that these meters should be neglected. They are very important, as it is always possible for leaks to develop which waste a large quantity of water. The large meters on commercial and industrial services should be read and billed monthly. On some installations, where the revenue runs into thousands of dollars a month, it is well to read the meter weekly, although no billing is made, to see that the meter is registering accurately.

The meter book is practically the same in all water departments. It has a separate sheet for each meter showing the customer's name and address, the make, size, number and location of the meter. Columns are provided for the date of reading, meter readings, consumption, amount of the bill, turn-on and -off readings and remarks. The book should contain approximately 200 accounts and a control set up for each meter book.

The meter reader is charged with the responsibility of reading the meter, determining the consumption by subtracting the previous reading from the present reading and checking with previous reported consumptions to determine if the present consumption is out of line, either high or low. If it is out of line he reads the meter again to be sure he has read it correctly and makes a notation in the remarks column that the reading is correct. Turning in correct readings is most important, as it eliminates incorrect bills and customer's complaints. In the remarks column the meter reader also makes a notation of meters not registering, meters leaking or requiring repairs, service leaks, meter boxes needing to be raised or lowered and any other condition that should be brought

to the attention of the department. This is what is known as "straight" meter reading, no inspection of the customer's premises being made at this time.

In reading meters located at the curb the meter reader should have a helper to take the top off the meter box and clean the glass of dirt and water, so that the meter can be read correctly. The number of meters that can be read by a meter reader with a helper varies from 300 meters per day, where the meters are located on paved sidewalks and the houses close together, to 200 meters per day in residential and rural sections where the meters become covered with grass and dirt and the houses are further apart.

Inspecting Customer's Premises

The meter reader does not have time to stop reading to inspect the customer's premises, as this would hold up the regular routine of getting the bills out on time. Inspection should be carried out by men trained and equipped for this work.

The inspector should get his orders promptly and follow right behind the meter reader. If this is done, inspections can be made before the customer gets the bill. Thus leaks can be detected, the waste of water stopped and complaints held to a minimum.

Good will is built up between the customer and the department because the customer feels that the department had enough interest in him to determine, without any request on his part, why the bill is high. The inspector's reports, along with his meter reading, should be entered on the meter sheet in the "Remarks" column so that the reason for the high bill can be explained to the customer and an applicable adjustment made.

Customer Accounting

When completed, the meter book is turned in to the office where the meter reader's computation of the consumption is verified and the water rate applied to the indicated consumption for each account. The consumption is compared with the consumption of previous quarters or months. If it is unusually high, an order is made to have the premises inspected. If it is low, an order is made to have the meter changed. Where the meter has stopped, the consumption is averaged, based on the previous use of water. Work orders are also made for the repair of all meters, leaks, etc., in accordance with notations made on the meter sheets by the meter reader.

To facilitate making out these orders a signal tab is placed on the meter sheets requiring work orders when the accounts are being checked, so that after the meter book has been completed these accounts can be located quickly.

It requires considerable experience to analyze the accounts properly to determine where meters are slowing up and revenue is being lost. If meters are changed unnecessarily the expense of operating the meter department is increased unnecessarily.

The principal systems in customer accounting are the "stub" plan, the "register sheet" plan and the "ledger" plan. There are many addressing, bookkeeping and billing machines that will take care of any demand of customer or company. The important thing is to choose the plan and equipment that gives the greatest degree of accuracy, control and economy.

In the "stub" accounting system, where the original bill is made together with a cashier's stub and an accounting stub, the audit sheet is placed beneath

the carbon under the bill and the exact data placed on the bill are recorded on the audit sheet. The machine automatically collects the totals of the meter readings, consumption and charges for each audit sheet. To prove that the correct consumption has been billed, the total of the consumption column, plus the total of the previous meter reading column, should equal the total of the present meter reading column. To prove that the correct charges have been billed, the number of accounts in each rate bracket can be multiplied by the rate and totaled. This total should equal the total of the charges shown on the audit sheet. The sheets are then used for control and balancing purposes.

When the billing has been completed the accounting stubs are torn from the bills and form the customer's ledger. The stubs are punched on one end when printed and are fastened together by a small bolt so they can be turned around when paid. Thus, the paid bills are one one side and the unpaid bills on the other side. In this way the unpaid bills can be more easily controlled.

This plan provides the most expeditious, accurate and economical means for billing a large number of accounts at frequent intervals and for maintaining the proper control at all times.

Payments received by cashiers are represented by cashier stubs detached from the bills and are balanced by the cashiers and passed to the customer's ledger checks. The cashier's stubs are then matched up with the accounting stubs which are stamped "Paid" and turned around to the paid side of the ledger.

While the bill calls for payment within ten days usually about ten days more is allowed for payment of the bill before it is considered delinquent. Final notices are then prepared and given to a collector. This notice calls for payment within two days and, if not paid within that time, a shut-off order is issued. Very few services are shut off because of non-payment as the customer usually pays the bill rather than have the service discontinued. In cases where the water is actually shut off the bill is paid within a few days and the service restored.

Sometimes the customer disputes the bill, claiming the water was not used and that the meter runs too fast. These are always difficult cases and should be handled by a tactful person with a thorough knowledge of the accounting system. The customer's account should be examined to see if an error has been made in making out the bill and if an inspection of the customer's premises has been made. If the cause of the trouble has been reported, it can be explained to the customer. If no inspection has been made, an order for one should be made and, if necessary, the meter should be tested. Every effort should be made to locate the trouble and satisfy the customer.

The relationship between an institution and its customers is the governing factor in its success or failure. It is, therefore, necessary that each and every employee of the water department deal understandingly and tactfully in their daily contacts with the customers. To collect all water bills and still maintain the good will of the customers is an ideal which is well worth while.

TESTING AND REPAIRING OF METERS—*A. T. Lundberg*

Chief Engr., Arlington County Water Dept., Arlington, Va.

The matter of testing and repairing meters is a water works operation which is common to all water departments, large and small, the only difference lying in the number of meters which each department handles. In the "Index to the Proceedings, Journal and Other Publications of the American Water Works Association" will be found a number of references under the subject heading of "Meters—Testing and Repairing."

Oklahoma City has arrived at a time limit of seven years for safe registration of domestic meters (1). At the end of this period, the meter is due for testing and overhauling at the shop. A complete overhauling is recommended, with the meter taken apart, cleaned and then reassembled, and all worn parts replaced so that it will meet the final test and be available for resetting.

In Haddon Heights, N.J., a five-year cycle has been set up for testing $\frac{5}{8}$-, $\frac{3}{4}$- and 1-in. meters, regardless of registration, and oftener for larger sizes (2). Repairing is complete, with special emphasis on the correct fitting of the disc in the measuring chamber. Accuracy of the meter at low flows is stressed.

The Illinois Commerce Commission has a rule which requires testing a $\frac{5}{8}$-in. meter every ten years or every 100,000 cu.ft.; $\frac{3}{4}$-in. meters every eight years or 150,000 cu.ft.; 1-in. meters every six years or 300,000 cu.ft.; and above 1-in. every four years (3). Again accuracy at low flow is emphasized. In repairing meters a complete overhauling is recommended, with replacement of all parts showing any wear.

The Waukesha, Wis., Water Department has arrived at a seven-year, 600,000-gal. limit for $\frac{5}{8}$-in. meters at which time they are brought in for testing and repair. Low-flow accuracy is once more stressed (4).

The average time limit for scheduling testing and repairing of the smaller-size meters seems to be from five to seven years and from 600,000- to 750,000-gal. registration. In the repair of meters it appears that special attention is given to accurate registration on the lower flows.

To accomplish these objectives, the testing and repairing program must be the responsibility of a capable person with sufficient help and a well-equipped shop. In some cases this program may be operated by one man, a single meter testing unit and the necessary tools and repair parts. An elaborate shop does not necessarily mean the best kind of workmanship.

Meter Maintenance in Arlington Co.

The Arlington County Water Department has been in operation since 1927 and has 228 mi. of water main and 16,053 service connections, 8,863, or 55 per cent, of which have been installed since 1937. A schedule of testing and repairing meters on a cycle basis as such has not been put into effect. The regular operation, however, partially covers such a program.

Meters reach the shop as a result of (1) discontinued service or cut-off, (2) non-registration found by meter reader, or (3) excess complaint by the consumer.

The practice is to remove the meter when service is discontinued or cut off. When the meter is brought to the shop

the registration is recorded on the removal card and forwarded to the billing office. The work on the meter depends on the length of time it has been in service. Some meters are practically new and need no repair but are run through the 1-gpm. test. When a meter has a large registration or has been in service for some time, it is taken apart, thoroughly cleaned, repaired and tested. The test rates are 10, 1 and 0.25 gpm. and the meter must register within 2 per cent before it is passed for use as a reset. The testing equipment consists of a Testerate indicator with provision for testing six meters at the same time.

Meters reported by the meter readers as not registering are taken care of by a removal order through the shop. They are routine through shop procedure. Domestic meters are read quarterly and commercial meters monthly, which limits the loss of registration to three months on the first group and one month on the second group.

Present practice provides for testing any meter for any customer when there is a reasonable doubt as to the amount of water used. The meter is removed and tested and a report sent to the customer. This report gives the actual results of the test.

The matter of new meters and testing is taken care of by spot testing, with each meter selected being tested on all three flows. For a more complete operation, a program of cycle testing and repairing is planned as soon as conditions permit.

The loss of revenue caused by the low registration of meters is often the factor which prompts the creation of sufficient funds for good operation. An inadvertent effect of under-registration is the unauthorized reduction in water rates. Most water departments have a definite rate structure under which water is distributed. When a meter that is under-registering is allowed to remain in service, water is automatically being furnished to someone at a reduced rate, which in most cases is not the intention nor does it come within the jurisdiction of the water department.

The problem of testing and repairing meters is one of utmost importance and too much cannot be said of its value. More and more water departments are becoming 100 per cent metered and this phase of operation definitely affects the water department's source of funds with which to carry on its work.

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Los Angeles Distribution System Flow Analysis Methods

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Presented on Oct. 26, 1944, at the California Section Meeting, Los Angeles, Calif.

THE Los Angeles water system has grown to be just a bit complicated during the past 40 years. In 1905 when the city purchased the water works system, the population of Los Angeles was approximately 250,000; in 1944 it was estimated to be 1,750,000. In 1905 there were about 200 mi. of street mains; today there are 4,274 mi. of mains. In 1905 the supply came from the Los Angeles River through the high-gravity and the low-gravity conduits which served the main city areas. The higher foothill areas were supplied by several pumping plants and there was little or no storage of water in reservoirs. Today there are fifteen major reservoirs with a combined capacity of 400,000 acre-ft., 75 minor reservoirs and tanks and 58 pump plants.

In 1905 the Underwriters' engineers were able to make a complete investigation of the system in several weeks. Nowadays such an investigation requires several months. In 1905 the operation of the system was comparatively simple in contrast to the operation of today. Because the increasing complexity of operation and design was recognized, the design engineers some years ago began planning a new way of recording and disseminating the basic facts of operation of the system so that such could be used most readily by the engineers.

Accordingly, a series of operation and design records was created and is maintained in a regular manner. Each week the significant features of operation are shown on such records and the charts are reproduced and distributed to the division and section heads for guidance in operation and design.

Weekly Flow Data Records

For example, Figs. 1 and 2 show two of the three weekly flow data records or sheets. The data on these sheets are obtained from the flowmeter and pressure-gaging station charts. The charts are removed once each week from the recording meters and the data are abstracted therefrom by the engineering division. The significant figures so obtained are lettered on a tracing. These sheets may be described simply as trunk line maps of the system, on which are shown the significant conditions of flow and pressures obtaining in the trunk lines and the water levels obtaining in the reservoirs during a seven-day period. In the rectangular blocks in Fig. 3 are shown the average weekly flow and the peak-hour flow for each trunk line, and in the circles the pressures or hydraulic grades at the peak hour at critical stations in the system. Because of the extraordinary physical extent of the Los Angeles system, it is necessary that three sheets be prepared in order

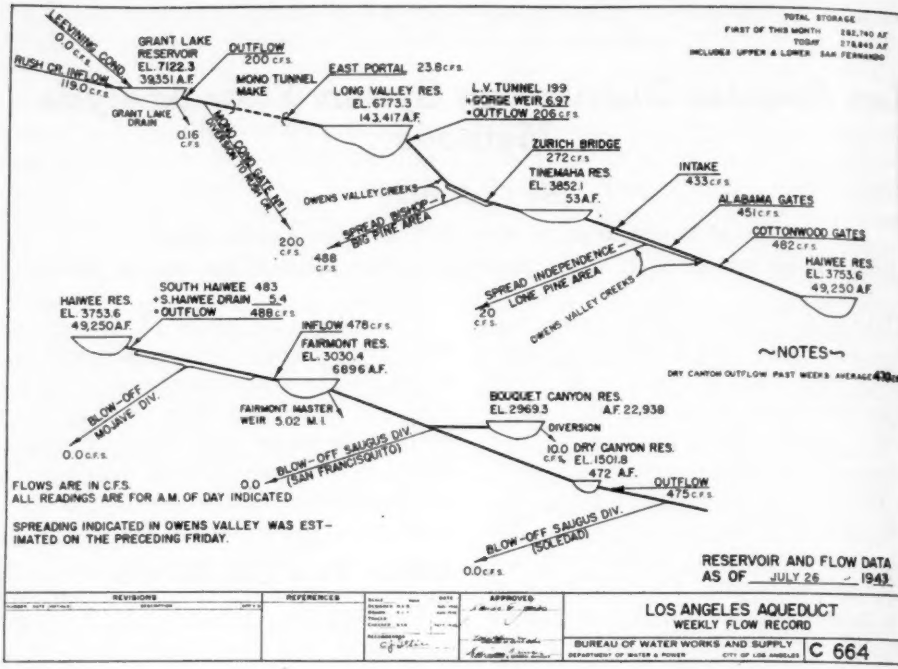


FIGURE 1 *

to show the entire system. One sheet shows the Owens River Aqueduct system, the second sheet shows all of the San Fernando Valley and the third shows the area of the city south and east of the Santa Monica Mountains, including the Los Angeles River works and the harbor distribution system; it also shows the Los Angeles trunk lines of the Metropolitan Water District.

Twenty sets of these flow records are issued weekly to water system executives and their assistants for use and aid in operation, sanitary control and design. The common use of such identical records helps greatly in the co-ordination of work and establishes a common understanding of operation among the divisions. Each person keeps a file of these records showing the weekly operation of the system, and this file proves a valuable reference throughout the year.

Annual Flow Data

At the end of each calendar year, another record, called the annual flow data record (Fig. 4), is made by noting the significant figures of flow, hydraulic grade and reservoir stage for the year. This sheet serves as a daily guide to the design engineers indicating the flows and pressures for the year just past, and assures that all future design of the system is based on the

* The author's illustrative material is reproduced in reduced size so as to convey to the readers the general information contained therein. Those persons who wish to make a study of the details are advised to request from the author full-scale prints of whatever material may be desired.

latest operational data available in the water system records.

This annual flow data sheet shows figures which are generally considered to be necessary in water system design (Fig. 5). For trunk lines are shown the mean yearly flow, the mean 24-hour flow on the day of maximum consumption and the flow at the peak hour of that day. For reservoirs are shown the stage or water surface elevation on the day of maximum draft and the lowest stage reached during the year. The hydraulic grades at key points in the system which obtained on the day and hour of maximum consumption are shown.

Most engineers can readily see the value of these records as a ready reference to the weekly and yearly operation of the system, but they may wonder as to their use in design.

System Design

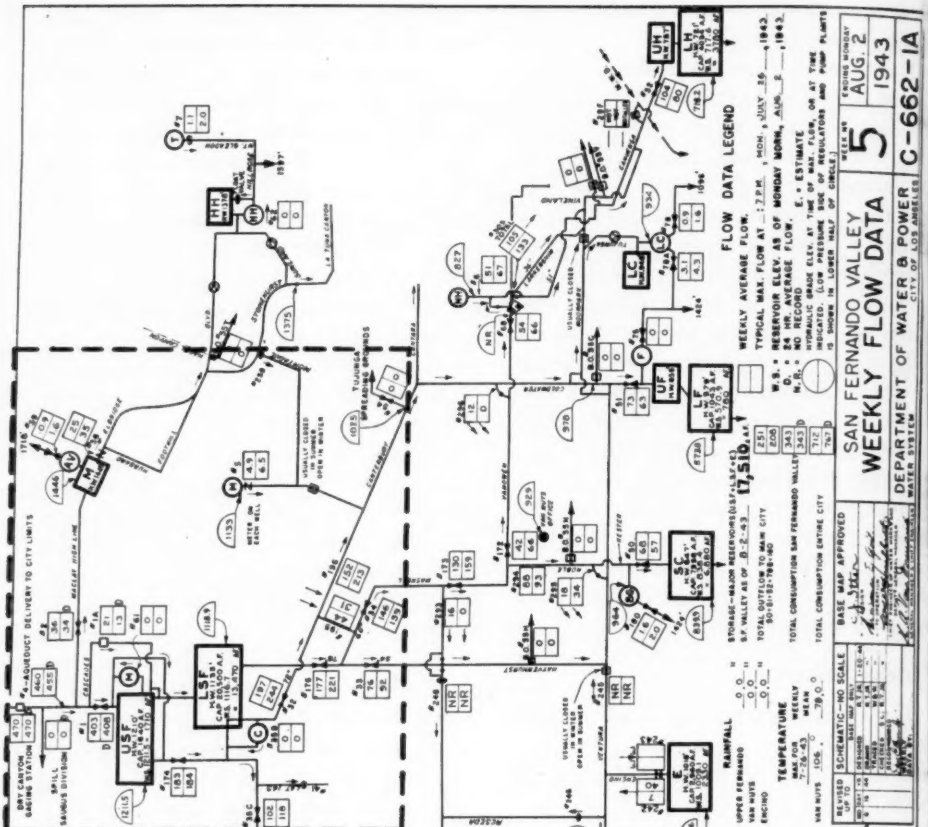
The term "design" as used in this paper means the planning of the extension of the system as one complete whole, fitting into one efficiently balanced unit the supply aqueducts, the reservoirs, the trunk lines and the distribution system. This paper is not concerned with the detailed structural or mechanical design of the particular works themselves.

Before preparing construction plans for the works considered, it is rather important to be able to show the need of such structures and their function in the water works system. It is a commonly accepted principle of engineering that before actual field work is done, an engineering report must be made to show why the work is needed in the first place and to give an explanation of how it is planned to function in the system at different periods of its life and under various assumed conditions of operation. Engineers have at various times admired a work of engineering and its monumental quality and then have been disturbed by some untutored ditch digger who says, "Wow! What a job! But what the devil is it good for?" The pyramids are magnificent engineering structures, but were much over-designed and have long outlived their usefulness for any purpose except as monuments to the ancient Egyptian dictators. The great Assuan dam across the Nile must seem of more value to the Egyptian taxpayers.

In America, where the taxpayers occasionally demand an accounting of the expenditure of public funds, engineers of municipally-owned water works must make plain the need for costly improvements and extensions to the system. These basic system design maps have been planned to do just that; that is, to show plainly on a water works system map the reasons trunk lines and reservoirs are needed and how they are to function at various times and under various conditions of operation throughout their estimated spans of life (Fig. 6). In themselves, these maps cannot be considered the equivalent of a comprehensive engineering report, but the engineers who use them find that it is quite easy to read on them the significant reasons of present operation and future design.

Basic System Design Map

The basic system design map (Fig. 6) is quite similar to the annual flow data record (Fig. 4). This is intentional. The design, sanitary and operating engineers become familiar with the weekly and annual flow data records by constant use and reference; and since the basic system design map


SAN FERNANDO VALLEY
WEEKLY FLOW DATA
DEPARTMENT OF WATER & POWER
WATER SYSTEM
CITY OF LOS ANGELES
FLOW DATA LEGEND
WEEKLY AVERAGE FLOW.
5
AUG. 2
1943
C-662-1A
SCHEMATIC—NO SCALE
BASE MAP APPROVED
RAINFALL
TEMPERATURE
UPPER FERNANDO
VAN NUYS
ENCINO
LEGEND
ITEM NUMBER
VENTURI METER
PITOT TUBE METER
SPARLING METER
OTHER METERS
GAGING STATION
RESERVOIR OR TANK
PUMPING PLANT
REGULATOR
BLOW OFF
SHUT GATE—CHECK VALVE
RELIEF VALVE
NOTE:
DRY CANYON GAGING STATION
SPILL
SAUGUS DIVISION
USF
LSF
TUJUNGA SPREADING GROUNDS
METER ON EACH WELL
USUALLY CLOSED
USUALLY CLOSED IN WINTER OPEN IN SUMMER
USUALLY CLOSED IN SUMMER OPEN IN WINTER
VAN NUYS OFFICE

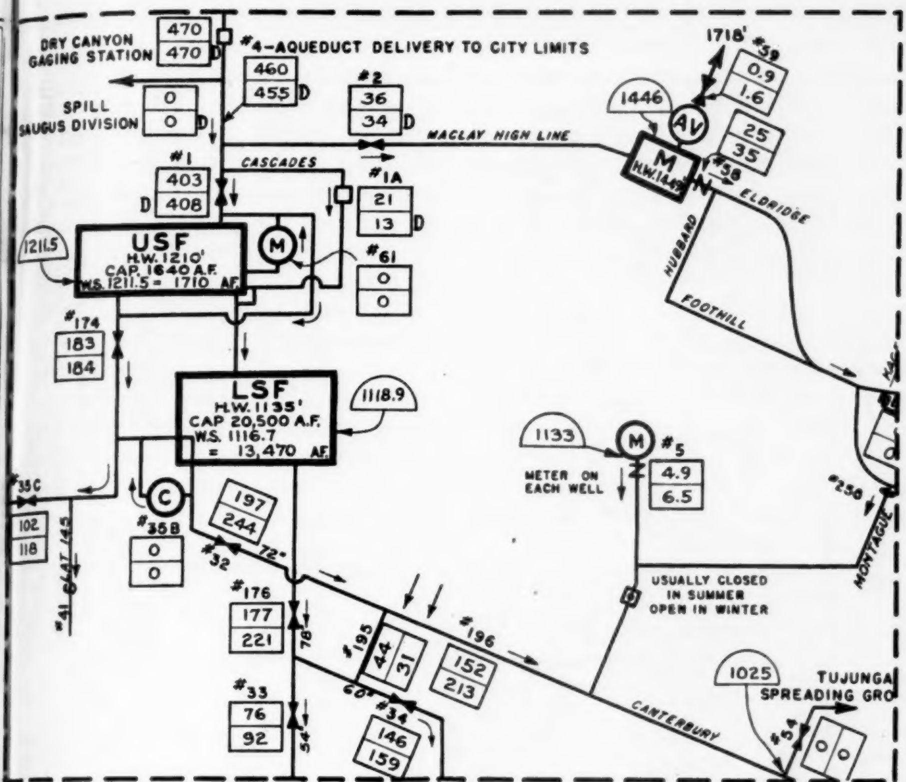
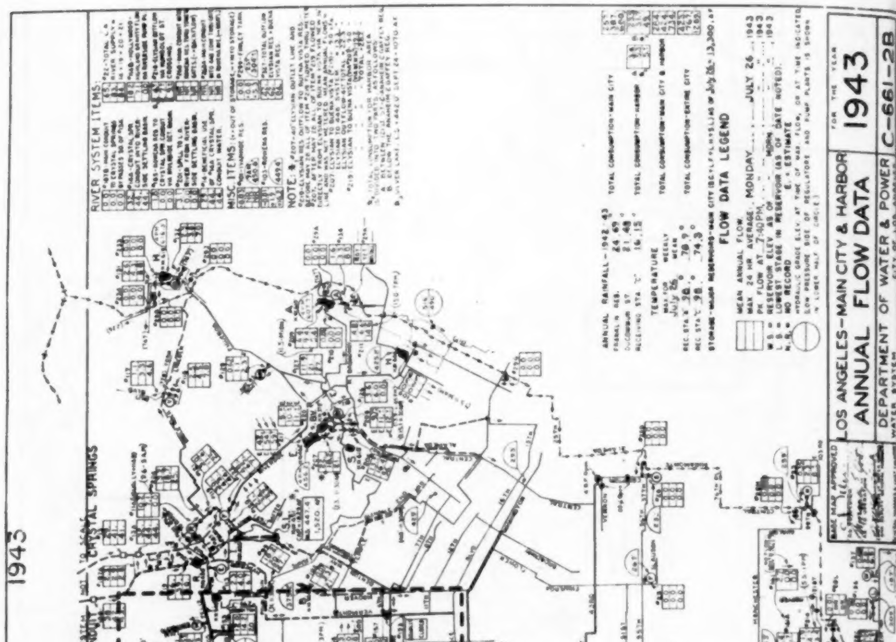


FIG. 3. Enlarged Detail of Portion of San Fernando Valley Weekly Flow Record

is also very similar to the weekly flow data record, it affords the engineers a well-known medium on which to check existing operation and to plan the system of the future. After looking at a hundred or more weekly editions of the flow charts over a period of several years, the design, sanitary and operating engineers find it not difficult to discuss their common problems over a familiar "Basic System Design Map" which is simply an annual flow data map accenting the future system design. This establishment and acceptance of a common form or map for use in considering the water system design and operation problems are invaluable to the co-ordination of work among busy executives and engineers and the establishment of good everyday working records is considered a good means of eliminating many unessential points of difference among engineers.

It will be noted from an inspection of the basic system design map (Fig. 7) that there is a block opposite each trunk line wherein are shown in the left-hand column the mean annual flow, the mean 24-hour flow on the day of maximum use and the maximum hour flow on the day of maximum use, as obtained throughout the last calendar year of record. Directly opposite these figures are other figures in the right half of the block, showing the


1943
MAIN CONDUIT
CRYSTAL SPRINGS
RIVER SYSTEM ITEMS
FLOW DATA LEGEND
LEGEND
LOS ANGELES—MAIN CITY & HARBOR
ANNUAL FLOW DATA
DEPARTMENT OF WATER & POWER
FOR THE YEAR
1943
C-661-2B

assigned *future* conditions of operating flows. This means that the design engineer plans future additions to the system with the basic facts of present operation directly in front of him. Since the design engineer submits his proposed future operation of the system directly opposite the existing or current operation of the system, it is almost impossible for him to forget or omit the consideration of some important factor of present or future operation in his design.

These sheets are sometimes referred to as water distribution balance sheets. That is to say, for some assumed condition of design, the water supply to the system must balance the water consumption. Therefore the use of these basic system design maps is a constant restraint on the under- or over-design of the water system, once the assumption of future total population to be supplied and the direction of the growth of the city have been established. Planning the improvements for a large water system by the use of these basic design sheets requires time and care, but it is well worth while, for when the map is completed, it remains as a definite record of how the system and the proposed works are to operate in the future, as compared with the operation of today.

For illustration, look at one of the sheets on which the Baldwin Hills

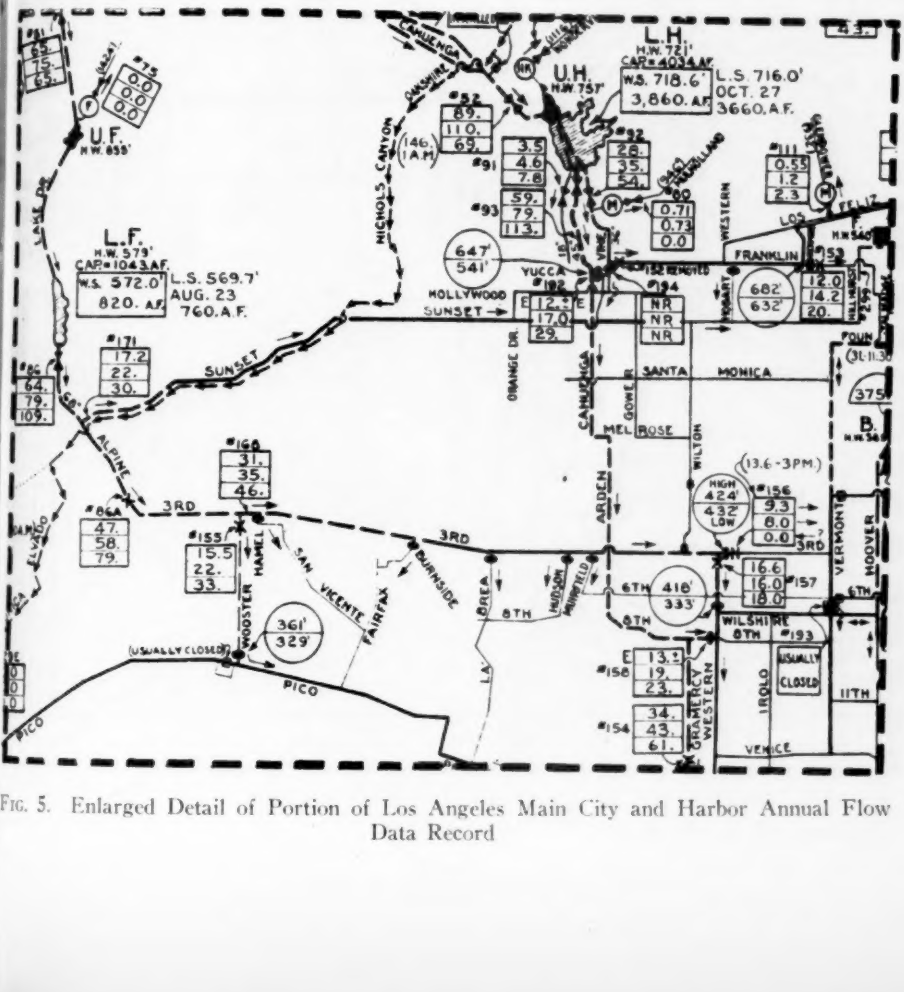
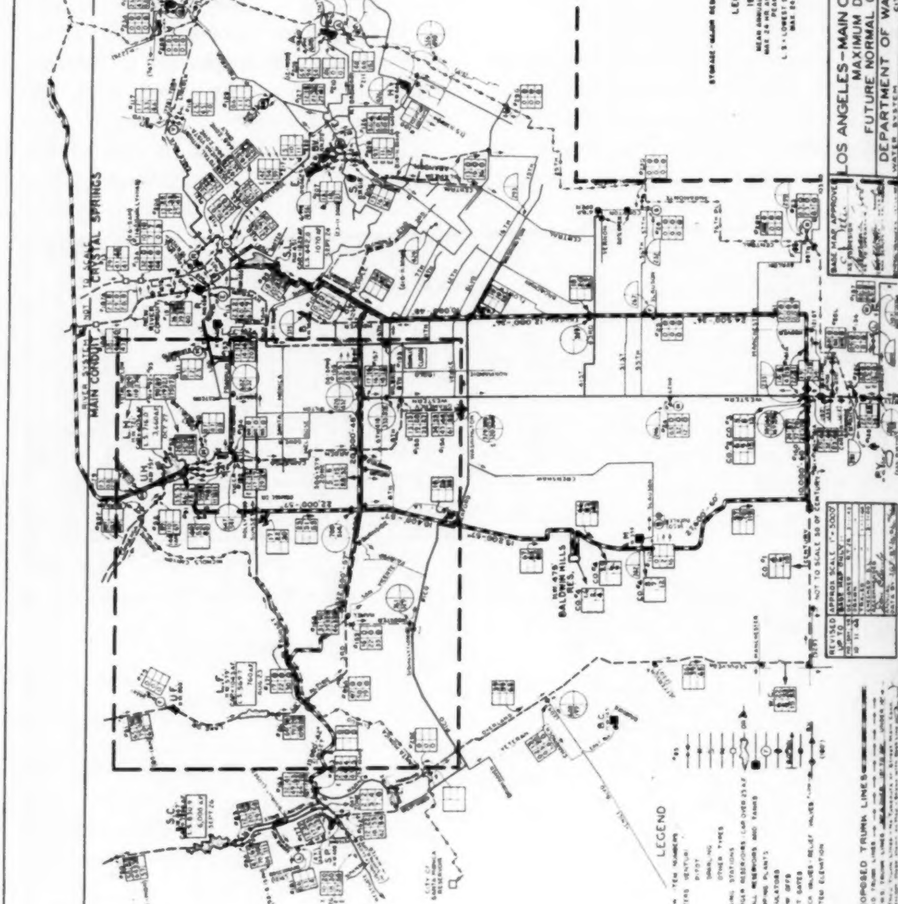


FIG. 5. Enlarged Detail of Portion of Los Angeles Main City and Harbor Annual Flow Data Record


LOS ANGELES—MAIN CITY & HARBOR
MAXIMUM DAY
FUTURE NORMAL OPERATION
DEPARTMENT OF WATER & POWER
BASIC SYSTEM
DESIGN MAP
1943 FLOW DATA
C-661-3A
RIVER SYSTEM ITEMS:
MISC. ITEMS:
LEGEND OF OPERATION
CRYSTAL SPRINGS
MAIN CONDUIT
BALDWIN HILLS RES.
LEGEND
PROPOSED TRUNK LINES

Project was planned (Fig. 6). The title in the lower right-hand corner and the legend above it state the basic conditions of design. In the left side of the flow blocks are the figures showing the existing use of water; the right half shows the assumed future use of water for the main city area under the three conditions of (1) future mean annual use, (2) the mean 24-hour use on the day of maximum use in summer and (3) the peak-hour use on the day of maximum use in summer. The locations of the Baldwin Hills Reservoir and the inlet and outlet lines are shown in Fig. 6, and adjacent to these lines are the flow blocks with the flow condition figures for which the lines are designed. The left-hand side of the flow block is free of figures, indicating that, since this is a proposed line and not yet a part of the system, no figures are available. A study of the figures on all existing trunk lines, however, indicates that both present operating conditions are shown in the left half of the block and future operating conditions governing an existing trunk line are shown in the right side. In this manner the basic system design map very clearly states that under the given conditions of design, as clearly set forth in the title and legend, the future operation of the system is to be as is shown plainly in the right half of the flow

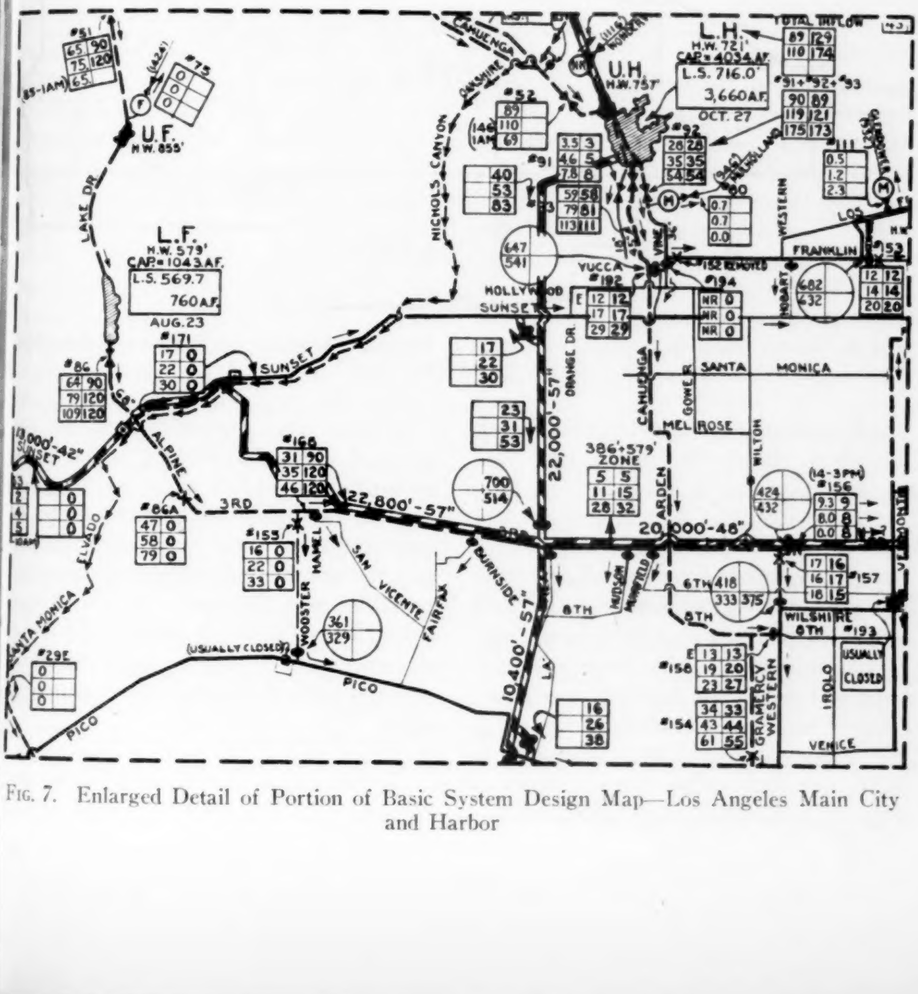


Fig. 7. Enlarged Detail of Portion of Basic System Design Map—Los Angeles Main City and Harbor

Elevation 579-ft. System Analysis—Los Angeles Water System

Based on October Water Consumers' Accounts by Meter Book Areas

579-System Meter Book Areas	Area *acres*	1939–1940			1940–1941			1941–1942			1942–1943		
		Bills Issued	Use *cfs.*	Duty *ft.*	Bills Issued	Use *cfs.*	Duty *ft.*	Bills Issued	Use *cfs.*	Duty *ft.*	Bills Issued	Use *cfs.*	Duty *ft.*
579–A	670	2,254	1.84	1.99	2,276	1.83	1.98	2,317	1.89	2.04	2,437	2.02	2.11
–B	1,312	4,795	4.32	2.38	4,798	4.07	2.25	5,130	4.29	2.37	5,116	4.43	2.4[illegible]
–C	1,222	3,741	6.17	3.65	3,911	5.89	3.49	4,145	7.67	4.54	4,138	7.61	4.51
–D	716	2,144	2.89	2.92	2,159	2.81	2.84	2,351	2.95	2.98	2,326	3.12	3.1[illegible]
–E	732	2,836	3.33	3.29	2,869	3.33	3.29	2,775	3.14	3.11	2,753	3.17	3.13
–F	885	2,642	2.78	2.28	2,658	2.87	2.35	2,693	3.01	2.46	2,687	2.89	2.36
–G	691	2,513	2.43	2.55	2,539	2.25	2.36	2,741	2.70	2.83	2,731	2.41	2.53
–H	705	2,553	2.50	2.57	2,683	2.34	2.40	2,642	2.61	2.68	2,629	2.82	2.90
Totals	6,933	23,478	26.26	2.74	23,893	25.39	2.65	24,794	28.26	2.95	24,817	28.47	2.97
Revenue:					$1,138,685			$1,265,969			$1,196,246		

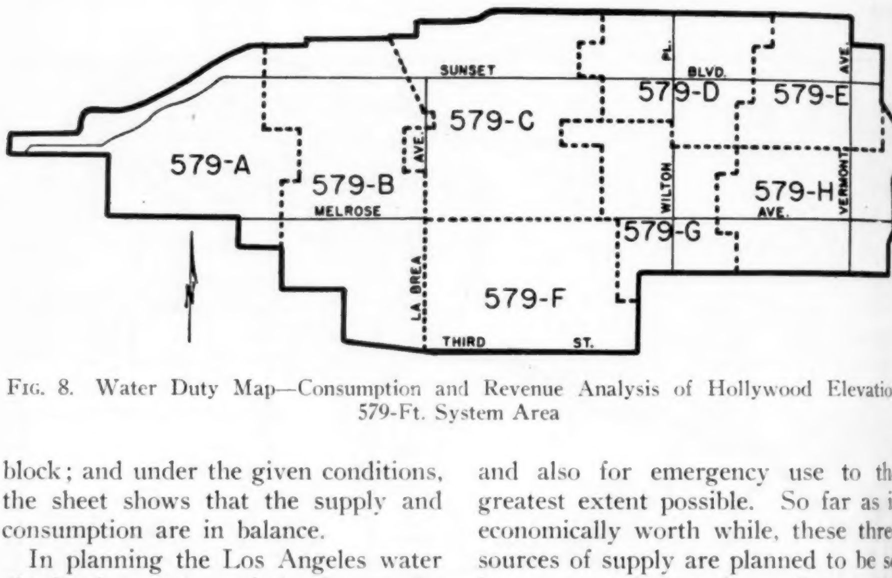


FIG. 8. Water Duty Map—Consumption and Revenue Analysis of Hollywood Elevation 579-Ft. System Area

block; and under the given conditions, the sheet shows that the supply and consumption are in balance.

In planning the Los Angeles water distribution system of the future, the design engineers are confronted with the problem of determining the best and most efficient and economical plan to put to use the Los Angeles River supply, the Owens River-Mono Basin supply and the Colorado River supply. The system is being planned to make available these three sources of supply for normal everyday future use and also for emergency use to the greatest extent possible. So far as is economically worth while, these three sources of supply are planned to be so interconnected that they offer to the city the greatest possible protection obtainable, and the basic system design maps form the work sheets which the design engineers use to plan the most efficient use of all three water supplies.

Water Duty Map

Before any water system can be designed, it is of course necessary to as-

sume the future use or duty of water over the area proposed to be served. In order to do this with the minimum amount of guesswork, the design engineers use a water duty map (Fig. 8), which is kept up to date by revision from year to year. The facts shown on this map are furnished from actual meter book records through the use of the tabulating machines of the department. The water duty map is a very important unit in the water system records and is constantly referred to for information concerning the principal growth and revenue figures. The entire map cannot be plainly shown in a small space, so only a particular district from this map is presented here. The district shown is well known as the Hollywood district. It shows an area known as the Elevation 579-ft. System Service Zone, so-called from the high water elevation of the Lower Franklin Reservoir which serves the area. This zone is subdivided into areas of from 1 to 2 sq.mi., shown as 579-A, 579-B, etc. These smaller areas comprise the territory covered by ten to twenty meter readers' books, and the tabulating machines are used to assemble the facts of bills issued, water consumption and water revenue. For instance, meter book area 579-A covers 670 acres. For the fiscal year 1939–1940, there were 2,254 monthly bills issued, there was a use of 1.84 second-ft., and the duty of water was 1.99 ft. In 1942–1943 in the same area there were 2,437 bills, a use of 2.02 second-ft., and a duty of 2.18 ft. This means that in this particular area the number of bills increased 183 in three years, the use increased 0.18 second-ft., and the duty increased 0.19 ft. The total water revenue derived from the 579-ft. service zone was $1,138,685 for the year 1940–1941 and $1,196,246 for the year 1942–1943—an increase of $57,561.

The district shown in Fig. 8 covers an area of 6,933 acres, or about 10.8 sq. mi. For many water works, this would be a large system in itself. In the same manner as described for this relatively small district, comparable statistics are secured for the entire water system of the city, giving the prime factors for interpreting and predicting the rate and direction of growth, which in turn becomes the basis of design for the system of the future. When it is considered that the actual area of the city now served by the municipal system is more than 350 sq.mi. and that the total area of the city is about 452 sq.mi., the analysis of growth and expansion of this rapidly growing water system would be a long, tedious and difficult problem without the use of this method of showing tabulating machine data on an adequate base map.

Good organization, design and operation of a large metropolitan water system demand that reliable facts be presented to the engineers and executives by means of a well-understood system of records. The quality of the daily operation and the future design of a large water system can be no better than the quality of the records on which they are based. Good planning and operation are directly dependent on the regular maintenance of a well thought-out set of operating records and, furthermore, on keeping those records constantly before the eyes of the design and operating engineers.

Design of Water Distribution Systems

By W. D. Hurst

City Engr., Winnipeg, Man., Can.

Presented on Apr. 21, 1944, at the Canadian Section Meeting, Niagara Falls, Ont., Can

THE design of distribution systems was somewhat static between 1900 and the advent of the Hardy Cross system of analysis in 1936, but since that time the subject has been a very live one.

It is not the author's purpose here to introduce any new conception of the art, but rather to correlate a number of the more important factors.

Distribution System Costs

The distribution system ordinarily costs much more than any other portion of a water works system, the cost varying from 55 to 70 per cent of the total, a commonly-accepted figure being 66⅔ per cent. The cost of mains and valves usually amounts to 80 per cent of the value of the distribution system. The value of the whole system as given by Howson (1) and based on nine recent water works valuations is approximately $200 per ton of pipe or, in another form, $63 per capita. Since approximately two-thirds of the value of the whole system is in the distribution system, it evidently has a value equivalent to about $40 per capita. It is therefore obvious that for economic reasons alone very great care should be taken in its design or redesign.

An engineer is rarely called upon to design a water distribution system to serve a community where no such system existed in the past, but nowadays is asked to design extensions or improvements to an existing system. Moreover, existing systems were not designed in the modern sense, and in some cases, like "Topsy," just grew. As the years passed, the locations of greatest demand changed and the communities expanded, requiring extensions to be made. Also in many cases the population became concentrated, resulting in sharply increased demands on feeders and sub-feeders.

Design of Water Main Grid

Water works distribution systems should be designed to supply two classes of service, namely:

1. Everyday domestic, commercial and public use of water, hereinafter referred to as ordinary flow or use.
2. Water for fire extinguishment.

In the first class the consumption is relatively uniform over the area served and is well distributed over 24 hours of the day, but in the second class the rate of use is high and is confined to a comparatively small area.

The first questions to be faced are: How much water is required to be transported? What are the average and maximum rates of flow to be expected? In order to answer these questions extensive data must first be gathered.

It is usually necessary to prepare a map of the area to be served at present, together with the area which may have to be served during the succeeding 25 years. A scale of 200 ft. to the inch is usually satisfactory for this purpose. This map should have contour lines at 5-ft. intervals and should show the division of the area into zoned residental, commercial and industrial districts. It should also show the location of all important buildings, manufacturing plants, railways, terminals, schools, libraries, large stores and all institutions housing large numbers of people. Moreover, the map should extend to the pumping station site and should show the source of supply, the pumping station and reservoir system.

A population study should be made with predictions for the next 25 years. It is here that good judgment is required, as over-optimistic estimates in the past have caused the unnecessary expenditure of large sums of money.

The engineer is now in a position to estimate water requirements. Average per capita metered consumption of water in the past has run from 40 gpd. for very small residential communities to 120 gpd. for large mixed communities. Of course unmetered average consumption runs very much higher—usually from 125 to 300 gpd. per capita. During recent years the figure of 100 gpd. per capita (83.3 Imp.gal.) has been commonly employed in the design of distribution systems for medium-sized and large mixed communities. The figures of 60 to 70 gpd. per capita of sixteen hours have been used widely in the design of systems for camps and cantonments for the armed forces.

Table 1 shows the quality of water required for ordinary use for various-sized communities.

TABLE 1

Water Required for Ordinary Use
(Based on 100 gpd. per capita)

Population	*gpm.*
1,000	70
5,000	347
10,000	694
25,000	1,735
50,000	3,470
100,000	6,940
200,000	13,880

With regard to peak loads to be expected, it has been found that water drawn on the maximum day of the year amounts to 150 per cent of that drawn on the average day, and that on the maximum hour of the day 50 per cent more water is drawn than on the average hour. Combining these, the ratio of maximum hour to average day is 2.25 to 1.00. For small communities this ratio may run as high as 3.00 to 1.00.

Table 2 shows the peak hourly loads that may be expected on the basis that peak hourly load = 2.25 × ordinary load as shown in Table 1.

TABLE 2

Peak Hourly Loads
(Based on 100 gpd. per capita)

Population	*gpm.*
1,000	158
5,000	781
10,000	1,560
25,000	3,900
50,000	7,800
100,000	15,600
200,000	31,200

The quantity of water required for extinguishment of fires has been discussed for many years. Some of the more common formulas for its determination are shown in Table 3. When determining the quantity of water and the number of fire streams required for any community in eastern Canada the Canadian Fire Underwrit-

TABLE 3

Quantity of Water and Number of Fire Streams Required as Established by Various Authorities

	John R. Freeman		Hazen & Kuichling	NBFU
	Maximum	*Minimum*		
Total quantity of water at fire stream of 250 gpm.	$Q = 250Y$	$Q = 25Y$	$Q = 250Y$	$Q = 1020(1.0 - .01\sqrt{X})\sqrt{X}$
Total number of fire streams required	$Y = X/5 + 10$	$Y = 1.7\sqrt{X} + 0.3X$	$Y = 2.8\sqrt{X}$	$Y = Q/250$

Where X = populatior in thousands, Y = total number of fire streams and Q = quantity in gallons.

ers Association should be consulted. In western Canada the Western Canada Insurance Underwriters Association is the appropriate body.

In recent years the formula of the National Board of Fire Underwriters has probably been used most extensively. Table 4 shows the water required for fighting fires as given by this formula for cities of various sizes.

TABLE 4

Water Required for Fire Fighting

Population	*gpm.*
1,000	1,000
5,000	2,250
10,000	3,000
25,000	4,750
50,000	6,500
100,000	9,000
200,000	12,000

The quantities listed above are in addition to water required for ordinary use. It will be seen that by comparing the cities in Table 1 and Table 4 having a population of approximately 200,000 persons or less the water required for extinguishing potential fires is greater than the average daily water requirements for ordinary use.

Because it is relatively unlikely that a major fire will occur at the time of the annual peak of ordinary use, it has been customary to provide for fire demand plus 150 per cent of the daily average ordinary demand (Table 5).

The quantities shown in Table 5 indicate the amount of water that the feeder mains must be capable of handling without excessive losses in pressure, provided no storage facilities can be made available. Figure 1 presents graphically the information tabulated in Tables 1, 2, 4 and 5.

TABLE 5

Required Fire Flow + 150 Per Cent of Average Daily Ordinary Flow

Population	*gpm.*
1,000	1,105
5,000	2,770
10,000	4,041
25,000	7,352
50,000	11,700
100,000	19,410
200,000	32,820

Storage Facilities

Inasmuch as the distribution system represents such a large portion of the cost of a water works system, the engineer designing it should very carefully consider the advisability of providing either underground or elevated storage or both. If distribution reservoirs are properly located it will be found that the first cost of pumping stations and filtration plants, as well as transmission and distribution mains, will be lessened materially.

In existing systems the provision of elevated storage will often be much more economical than the reinforcement of existing mains.

It should be emphasized, however, that the amount of water to be stored and the location and elevation of such storage are local problems and should

in each case be given individual study. Standpipes, of course, should be provided only where the natural elevation of the ground will make them effective in the development of head, as it is never considered economical to use a flat-bottom storage tank where the tank itself must provide the elevation to secure the proper pressure.

The data in Table 6, obtained through the courtesy of Louis R. Howson, give some idea of the weight of various sizes of steel standpipes.

TABLE 6

Weight of Steel Standpipes of Various Sizes

Capacity gal.	*Size* ft.	*Weight* lb.
374,000	24×110	220,000
612,000	40× 65	196,000
725,000	50× 50	260,000
850,000	50× 60	270,000
1,000,000	40×100	414,000
1,100,000	42×100	485,000
1,650,000	60× 80	620,000
1,650,000	75× 50	480,000
2,000,000*	75× 60	650,000
2,750,000*	85× 65	756,000

* Includes roof.

For a rough estimated cost of standpipes unit prices of 12 to 12½ cents per lb. of metal may be used for Canadian conditions and unit prices of 9 to 9½ cents per lb. for United States conditions.

Howson (1) has stated that if elevated storage of from 20 to 40 gal. per capita is provided the rate of pumping can be made constant throughout 24 hours of the day. Even with the provision of elevated storage of from 5 to 10 gal. per capita the peak rate of pumpage can be reduced by half its excess over the daily average. Howson further states that with elevated tank costs at 12 cents per lb. of steel the provision of such storage to the extent of 30 gal. per capita will cost only from 6.25 to 9 per cent of the value of the distribution system, depending on the population. Furthermore, the provision of such storage will enable the community to avoid high electric peak demand charges, to secure more uniform pressures, to protect against power outages and give better operation of pumping equipment, to say nothing of the saving in the cost of the distribution system.

Figures 2 and 3 show graphically the relationships to one another of capacity, height and estimated weight of elevated tanks. These graphs should be used only for rough estimations of weight. For accurate estimates of weight, companies specializing in elevated storage should be consulted. To obtain the approximate cost of elevated storage multiply the weight of the tank in pounds by 12 cents. In Canada, the weight of the tank in pounds is multiplied by 15 cents.

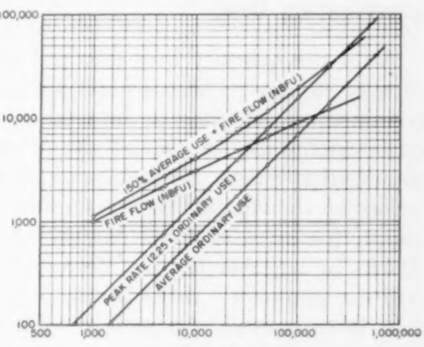


FIG. 1. Water Required for Ordinary Use and for Fire Flow

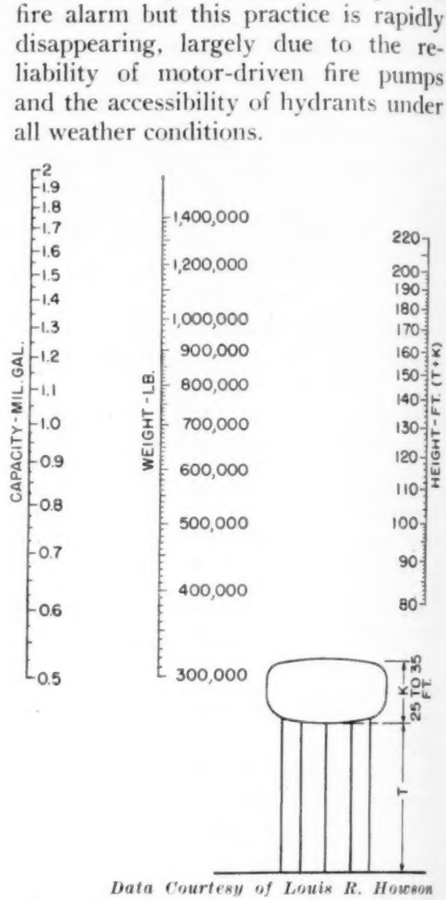
As fires are ordinarily of short duration, it is apparent that the cost of the distribution system may be reduced by the provision of reservoir storage to meet the fire demand rate for periods of ten hours, as suggested by the Underwriters. Howson (1) has estimated that reinforced concrete reservoir storage to meet the fire demand

for ten hours can be provided for 27.5 per cent of the distribution system value for communities of 5,000 persons to as low as 1.97 per cent for communities of 200,000 persons. The location of such reservoir storage and its effect on the valuation of the distribution system are local problems and must be solved individually. Its effect on the total valuation of the systems will be greatest when topographical conditions permit the storage to be at such an elevation that it will float on the distribution system. Its value will be less when it is located at approximately the same ground level as the pumping station but at the far end of the distribution system, and still less when it is located adjacent to the pumping station. Elevated storage to provide for ten-hour fire use is very costly and is usually not economical.

Distribution Grid

After the quantity of water and the type of storage to be provided are determined, the engineer is in a position to determine the layout and size of the distribution grid. The question now arises as to whether the system is to supply fire streams direct from the hydrants or whether the fire department will use pumpers. If the former situation is to prevail, the grid should be designed so that not less than 50 psi. of pressure shall be obtained at the hydrant. If, however, pumpers are available, a pressure of 20 psi. at the hydrants is sufficient to feed medium-sized pumpers of, say, 750-gpm. capacity. A pressure of 10 psi. is sufficient for large pumpers having a capacity of from 1,000 to 2,000 gpm. It is usually inadvisable because of plumbing considerations to carry pressures on the mains of over 125 psi., and, in general, pressures of from 70 to 100 psi. are more satisfactory.

Formerly it was quite common practice to boost pressures on receipt of a fire alarm but this practice is rapidly disappearing, largely due to the reliability of motor-driven fire pumps and the accessibility of hydrants under all weather conditions.



Data Courtesy of Louis R. Howson

Fig. 2. Nomograph for Approximate Values of Capacity, Weight and Height of Radial-Cone Type Elevated Tanks

The distribution system can be divided into three general classes:

1. The primary feeders, consisting of a skeleton of large pipes which convey large quantities of water to points on the distribution system for local distribution.

2. The secondary feeder network, consisting of pipes of intermediate size

which reinforce the primary feeders, aiding in the concentration of the fire flow at any point in the system.

3. The distributors, which consist of a grid of small mains feeding hydrants and blocks of consumers.

Three further general systems have been employed in the layout of the distribution network: (1) the ring system, (2) the central main plan, and (3) the intermediate plan.

The "ring" system consists of a large main around the periphery of the area to be supplied and a gridiron of laterals extends from this outside main across the district.

The "central main" plan consists of a large main laid through the center of the district with laterals branching from it to form a network reaching all parts of the district.

The "intermediate" plan combines the best features of the ring plan and the central main plan and consists of an interior ring in the network not extending to its outer boundaries.

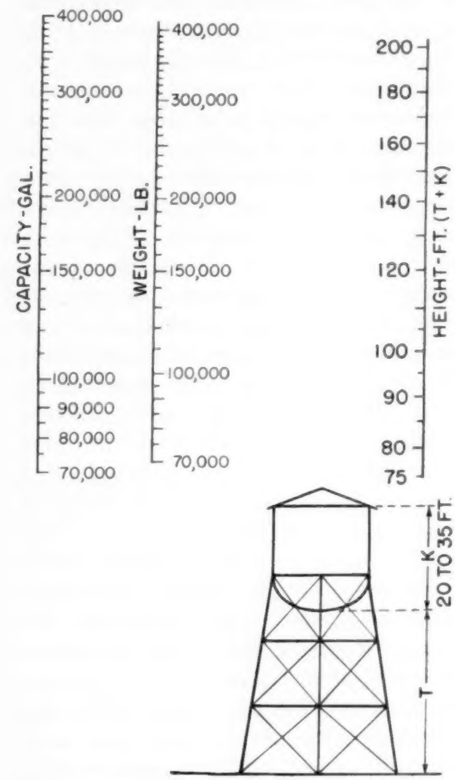
Rudolph Hering (2) investigated the merits of the three systems proposed and concluded that either the central main plan or the intermediate plan was more economical than the ring system and he showed that a "layman engineer" might innocently add about one-third to the cost of a distribution system without gaining any advantage in efficiency.

In the distribution system two types of pressure drop are usually encountered: (1) the loss of head in the primary feeders, which varies in general with the total draft; and (2) the loss of head within a relatively small area surrounding the fire, due to concentration of fire flow.

Keeping the above principles in mind, a trial layout should be made and subjected to analysis. A good start is to provide, if possible, two or more feeder mains from the main pumping station or distribution reservoir, as the case may be, running to or near the center of gravity of consumption. Large mains should be provided at intervals of from $\frac{1}{4}$ to $\frac{1}{2}$ mi. apart in both directions and these areas filled in with smaller pipes to form a grid. As to capacity it is safe practice to design the feeder system to convey the maximum supply to an area with the allowable loss of head, the contribution of the smaller pipes being taken as a margin of safety as well as an offset to loss of capacity of pipes as they become older. This loss of capacity of tar-coated cast-iron pipes as they age was fully considered by a committee of the New England Water Works Association who reported in 1935 that the average actual loss in capacity of tar-coated cast-iron pipe after 30 years of service, based on a total of 473 tests in nineteen different systems, was 52 per cent.

The sizes of smaller pipes which form the grid are usually tentatively determined from past experience, but depend upon the number of fire streams required at any given point. A common arrangement in small cities and the outer districts of large cities consists of having 6-in. cross mains with 8-, 10- or 12-in. pipes at intervals of from four to six blocks apart. Some cities do not now employ mains of less than 8 in. in the grid system and most cities have entirely abandoned the use of 4-in. pipe in the grid. A 4-in. pipe is usually quite satisfactory for supplying water for ordinary use, but unsatisfactory for supplying it for fire use through hydrants. For heavily built-up compact districts the grid should be filled in with 8- to 12-in. pipes in one direction, with 12- to 16-

in. diameter cross pipes at frequent intervals. The feeders, of course, will be much larger. A satisfactory approach to the determination of the size and arrangement of the small mains is the circle method, described in the Appendix hereto.



Data Courtesy of Louis R. Howson

Fig. 3. Nomograph for Approximate Values of Capacity, Weight and Height of Ellipsoidal-Bottom Type Elevated Tanks

It is very important that all pipes be connected where they cross. It can be shown that if the pipes are all one size a given quantity of water can be delivered by a hydrant by two, three or four routes with respectively one-half, one-third and one-quarter the velocity in a single pipe and with a corresponding loss in friction of one-quarter, one-ninth and one-sixteenth of that in the single pipe.

Freeman (3) has stated: "So far as a general statement may apply, we should say that the pipes should be large enough so that two-thirds of the total number of fire streams could be concentrated on one block in the compact valuable part of a city or upon any one large building of special hazard."

Table 7 shows the percentages of the various sizes of mains installed in selected Canadian municipalities. The percentages tally fairly closely with United States practice as reported by Howson (1).

Hydraulic Calculations

In Canada and the United States the flow formula which has found the widest application for water works use is the one devised by Hazen and Williams, namely:

$$V = C R^{0.63} S^{0.54} 0.001^{-0.04}$$

With regard to the value of the coefficient C the following values have been commonly used:

Reinforced-concrete glazed-interior pipes and asbestos-cement pipe	$C = 130-140$
Large cement-lined cast-iron and steel pipe	$C = 130-140$
Large unlined cast-iron and steel pipe	$C = 130$
Small cement-lined cast-iron pipe	$C = 120-130$
Small unlined cast-iron pipe	$C = 100-120$

When using lined cast-iron pipe or asbestos-cement pipe probably a safe value for C would be 120, which value would allow sufficient loss of carrying capacity, considering the type of material.

When using unlined cast-iron pipe probably the most common value used is $C = 100$, except in regions where

TABLE 7

Percentage of the Various Sizes of Water Mains Installed in Selected Canadian Municipalities (Taken From "Water Works Information Exchange," 1936)

Municipality	Percentage of Total Length of Tabulated Diameters of Mains												
	4-in.	5-in.	6-in.	8-in.	10-in.	12-in.	14-in.	16-in.	18-in.	20-in.	24-in.	30-in.	Other Sizes
Victoria, B.C.	47.2	2.6	21.1	8.9	3.4	11.8				1.9	0.4		2.7
Vancouver, B.C.	26.1		40.5	15.6	1.4	9.2	0.7	0.9	1.2	0.2	1.3	0.2	2.7
Calgary, Alta.	2.5		64.0	3.5	8.9	8.9		1.9	1.4	3.4	1.6	0.2	3.7
Regina, Sask.	0.4		71.5	15.2	3.5	0.3		3.0	2.2		4.1		
Winnipeg, Man.	3.4	0.2	54.0	20.3	9.0	5.2	2.7	1.2	3.2	0.2			0.5
Port Arthur, Ont.	11.3		61.2	7.9	0.4	6.5					12.7		
Windsor, Ont.	18.7		45.8	15.4	3.3	11.5		2.1		2.2			
London, Ont.	35.4		42.3	6.2	6.9	4.3			4.8	0.1			
Hamilton, Ont.	1.2		71.5	1.3	1.0	10.8		0.7	2.6	4.6	1.8	3.7	1.0
Toronto, Ont.	2.1		68.5	1.3	0.7	19.3		1.0		1.4	2.1	0.6	2.9
Peterborough, Ont.	11.2	38.0	15.3	18.6	2.3	6.7	1.1	3.2	3.5	0.1			
Kingston, Ont.	9.7		77.8	10.3	0.2	2.0							
Niagara Falls, Ont.	14.2		61.4	1.2	4.8	8.8			1.9	0.6	1.6		5.5
Ottawa, Ont.	4.0	33.1	32.1	11.6	1.0	8.1		3.9	0.6	0.4	1.3	0.2	3.7
Montreal, Que.	8.3		16.4	42.0	7.1	13.4	0.4	2.8		1.5	2.5	2.2	3.6
Lachine, Que.	12.1		48.2	7.9	11.5	11.5	3.9	0.4		3.7		0.2	0.5
St. Stephen, N.B.	1.0		50.5	7.9	7.4		6.2	27.0					
Charlottetown, P.E.I.	14.2		42.5	6.8	4.6	7.9	24.0						
AVERAGE OF ALL MUNICIPALITIES	12.4	4.1	49.1	11.2	4.3	8.1	2.2	2.6	1.2	1.1	1.6	0.4	1.5

tuberculation is known to be severe. In these instances probably a value of $C = 80$ would be safest.

The system can now be analyzed so as to determine the amount and direction of flow through the mains and the head losses up to any point in the grid. The analysis is accomplished by a method of successive approximations devised by Professor Hardy Cross in the 1930's. It is beyond the scope of this paper to outline the details of the method which are fully covered in the original paper (4), in the technical press (5) and in a previous paper (6), in which this author had a share. It was pointed out in the latter paper (6) that one of the principal problems of application of the Hardy Cross method was the determination of the points of "take-off," but once having decided this it is a mechanical matter to solve the network.

Upon solution, a contour pressure map can be prepared and the contours compared with those of a theoretically perfect system. The contour plan will show the weaknesses of the network and larger mains can be substituted where needed or decreased in size when necessary and the network reanalyzed on the basis of the changed pipe sizes. The network can then be analyzed for fire flow. A fire should be assumed somewhere in the congested value district, and two-thirds of the total fire streams required for the city assumed to be concentrated in the block in which the fire occurs and drawn from the nearest hydrants to the fire. With these take-offs assumed the network can be re-analyzed by the Hardy Cross method to see whether the necessary water can be fed by the mains so that the pressure losses at the hydrants are not excessive.

The preceding description of the determination of the size and layout of the distributing system has reference to a new system but the same principles apply to the re-design of an existing system and differ only in the application.

Physical Nature of the System

Before the size of pipe in the distribution system can be determined the engineer should decide on the type of mains that will best suit the community in which they are installed. Having done this the coefficient *C* for use in the analysis can be selected.

Mains are subject to two kinds of attack: (1) from the water which they carry and (2) from the material in which they are embedded.

The kinds of pipe that have been normally used in distribution system feeders and laterals are:

1. Cast-iron pipe
2. Steel pipe
3. Reinforced concrete pipe
4. Asbestos-cement pipe
5. Wood stave pipe

Cast-iron pipe has been the most widely used, both for feeders and laterals. Steel and reinforced concrete have been used principally for feeders, and asbestos-cement and wood stave for both feeders and laterals.

The interior of metal pipe has suffered serious tuberculation in certain areas of the country, which has caused loss in carrying capacity with the passage of time. The Committee on Pipeline Friction Coefficients of the N.E. W.W.A. studied this matter and summarized the effect of hydrogen-ion concentration values of the water upon carrying capacity as shown in Table 8.

This is a very serious matter and insufficient attention has been given to this problem in the past. Of course

TABLE 8

Effect of pH Values of Water Carried Upon Capacity Loss After 30 Years of Service

pH	*Approximate Average Capacity Loss, %*
8.0	30
7.5	35
7.0	45
6.5	60
6.0	85

water main cleaning and scraping can restore a large portion of the loss, but the cleaning of entire distribution systems is not usually attempted and the better plan is to foresee this trouble and provide against it. Cement lining of cast-iron and steel pipes is now very much more popular and should be provided where cast-iron or steel pipe is to be used under moderate to severe tuberculation conditions. Asbestos-cement pipe and reinforced concrete and wood stave pipe, being non-metallic, do not suffer from this trouble.

The other problem of external attack of metallic pipe has also been severe, particularly in the mid-western and southern portions of the United States and Canada. This attack is apparently caused by both acid and alkali soil conditions. The pipe becomes graphitic and the iron can be cut with a pen-knife. Figures 4 and 5 show pieces of cast-iron pipe removed from the Winnipeg distribution system which have been attacked by both soil corrosion and electrolysis. Various coatings have been tried in Winnipeg with varying success (usually very little). The answer to this problem seems to be to surround the pipe with a material not subject to alkali or acid action (such as alkali-resisting concrete), or to provide under-drainage to remove the moisture necessary to continue the action of the attack, or to use non-metallic pipes. The steel

bands of wood stave pipe are, of course, attacked in the same manner.

As the piping system grows older it usually becomes necessary to retire portions of it because of such factors as corrosion, insufficient carrying capacity, relocation of streets, etc. A study of the reasons for and the amount of pipe retired from the streets of the city of Winnipeg was made during 1943, the results of which are shown in Table 9. It will be noticed that 75 per cent of all main retirements have been due to physical conditions, which is a rather unusual situation due to corrosive soil conditions in the district. Functional causes, such as the action of public authorities, have been the more important reasons for retirement in most Canadian and American cities.

When specifying cast-iron and steel pipe the following specifications will insure the purchaser protection against inferior products and workmanship:

1. American Standards Association Specifications for Cast-Iron Pit-Cast

TABLE 9

Installation and Retirement of Distribution Mains
City of Winnipeg

Size in.	Kind	Installed	In Service	Retired	Approximate Percentage Retired	Percentage Retired Classified as to Reasons Retired			
						1A*	2A*	2B*	2C*
3	Cast-iron	199	199	0	0.00				
4	Cast-iron	195,136	134,645	60,491	31.00	25.79	0.22	4.25	0.74
5	Cast-iron	14,775	5,770	9,005	60.90	60.90			
6	Cast-iron	953,475	926,375	27,100	2.84	1.75	0.11	0.41	0.57
6	Asbestos-cement	2,454	2,454	0	0.00				
8	Cast-iron	244,035	223,179	20,856	8.55	6.32	0.12	1.54	0.57
10	Cast-iron	163,879	163,347	532	0.32	0.03		0.21	0.08
10	Flexible joint (c.-i.)	839	839	0	0.00				
10	Asbestos-cement	8,854	8,854	0	0.00				
12	Cast-iron	95,150	91,692	3,458	3.63			0.77	2.86
12	Flexible joint (c.-i.)	782	782	0	0.00				
14	Cast-iron	40,268	40,223	45	0.11				0.11
14	Asbestos-cement	431	431	0	0.00				
16	Cast-iron	18,868	18,868	0	0.00				
18	Cast-iron	39,750	38,637	1,113	3.03				3.03
18	Asbestos-cement	859	859	0	0.00				
18	Victaulic joint (c.-i.)	113	113	0	0.00				
20	Cast-iron	4,200	4,052	148	3.52				3.52
24	Cast-iron	11	11	0	0.00				
24	Reinforced concrete	3,899	3,899	0	0.00				
36	Steel—Concrete covered	1,200	1,200	0	0.00				
	Total—Feet	1,789,177	1,666,429	122,748	6.86	5.11	0.10	0.95	0.70
	Miles	338.86	315.61	23.24					

* Reasons for retirement: 1, physical (1A, wear and tear); 2, functional (2A, inadequacy; 2B, obsolescence; 2C, action of public authorities).

Pipe for Water or Other Liquids—A.S.A. A21.2-1939.

2. United States Federal Government Specifications for Pipe: Water, Cast-Iron Bell-and-Spigot—WW-P-421. (Covers centrifugally-cast, horizontally-cast and Universal bolted-joint types.)

3. American Standards Association Specifications for Cement-Mortar Lining for Cast-Iron Pipes and Fittings—A.S.A. 21.4-1939.

4. American Water Works Association 1908 Standard Specifications for Cast-Iron Special Casting

5. American Water Works Association Standard Specifications for Laying Cast-Iron Pipe—7D.1-1938.

6. American Water Works Association Specifications 7A.1-1940; 7A.2-T; 7A.3-1940; 7A.4-1941-TR; 7A.5-1940; 7A.6-1940; 7A.7-1941. (Cover steel pipe and protective coatings.)

Standard specifications for asbestos-cement pipe have not yet been issued but in the meantime the city of Winnipeg requires the Underwriters' Approval Label to be attached to each length of pipe supplied.

Reinforced concrete pressure pipe should be required to meet the A.W.W.A. Tentative Emergency Specifications for Reinforced Concrete Pressure Pipe—7B-T.

Services

The materials most widely employed in the pipes serving houses and buildings are lead or copper. The use of iron pipe (black or galvanized) has been confined to localities where corrosive conditions are not present to any extent. The life of lead service pipe in the Winnipeg area runs from 30 to 40 years and the pipe is found to be very brittle and granular when removed from service. Copper pipe had been used in Winnipeg from 192[illegible] up to the time the use of this material was prohibited by the metals controller. This material has shown no signs of failure during this 20-year period. What its ultimate life is cannot be stated except that it exceeds 20 years in the Winnipeg area. Where very soft waters are to be carried in the piping system the use of lead pipe should be very carefully investigated to determine whether or not the lead is being dissolved in the water. If such conditions are present lead poisoning may result.

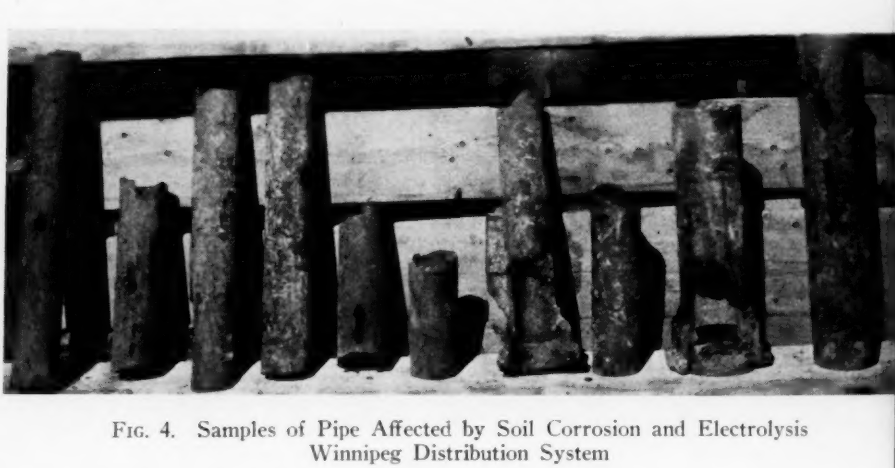


FIG. 4. Samples of Pipe Affected by Soil Corrosion and Electrolysis Winnipeg Distribution System

FIG. 5. Samples of Pipe Affected by Soil Corrosion and Electrolysis
Winnipeg Distribution System

The American Society for Testing Materials specifications for copper water tube (B88-41) and for steel pipe (A120-44) are satisfactory and materials supplied should meet these requirements. The U.S. Bureau of Standards Commercial Standard CS95-41 are satisfactory for guidance in the purchase of lead pipe.

In Canada the appropriate specifications are those issued by the Canadian Standards Association as follows:

B-66-1940—Standard Specifications for Copper Water Tube

B-67-1941—Standard Specifications for Lead Service Pipe

B-63-1942—Standard Specifications for Welded and Seamless Steel Pipe

Meters

The question of metering should be carefully studied and, unless very good reasons are advanced against meter installation, 100 per cent metering should be provided. It will be found that per capita consumption of water will be very nearly cut in half if this is done. In Chicago, where only from 25 to 30 per cent of the services are metered and where the average per capita consumption is 270 gpd., it has been estimated that universal metering would reduce the above-mentioned figure to 170 gpd.

Where meters are properly maintained it will be found that the unaccounted-for water, i.e., the difference between the pumpage and water paid for by meters, or otherwise accounted for, will amount to only from 10 to 20 per cent, whereas with infrequent maintenance it will amount to from 30 to 40 per cent. At Des Moines, Iowa, a few years ago the city was accounting for but 58 per cent of the water pumped. More attention was then given to meter maintenance and particularly to the ability to catch low flows, with the result that the city now accounts for 89 per cent of all water pumped.

Under the old policy in Winnipeg meters were removed only when they failed to register. Now the city has a regular system of removing and maintaining its meters. The large sizes (1-in. and over) are removed, checked and repaired, if necessary, on the average of once every two years. The smaller sizes ($\frac{5}{8}$- and $\frac{3}{4}$-in.) are removed on the average of once in seven years.

In the design of a new system serious consideration should be given to the provision of low-head-loss main line meters on the principal district feeder mains so that a more accurate analysis of the flow conditions can be made. The provision of district meters, in the author's opinion, is money very well spent.

Valves

In the operation of a water system one of the most important pieces of equipment is the distribution gate valve. Experience in Winnipeg has shown that far too few gate valves have been installed in the past. In extreme cases it has been necessary to operate as many as 25 valves in order to effect a shut-off. To provide four valves at each intersection would be going to the other extreme, but the city now tries to provide two valves at every intersection. The National Board of Fire Underwriters recommends a spacing in which "no single case of accident, breakage or repair to the pipe system, exclusive of arteries, will necessitate the shutting from service a length of pipe greater than 500 ft. in high-value districts or greater than 800 ft. in other sections, and will not result in shutting down an artery." The water mains and the valves should be arranged systematically so that they can be easily found. In Winnipeg the mains are laid on the east and south sides of the street on a line 14 ft. from the property line and the valves are located in range with the property line forming the intersecting street, except in long blocks when they are set also at the center of the block. All valves 12-in. and larger should be set in vaults so that repairs and re-packing can be easily handled. In fact it is the author's opinion that in cold climates no mistake is made in providing a vault for all valves, irrespective of size. This undoubtedly will add considerably to the capital cost of the system but results in much superior maintenance and resultant freedom from trouble. Gate valves should be required to meet the A.W.W.A. Standard Specifications for Gate Valves for Ordinary Water Works Service—7F.1-1939. In Winnipeg the provision of 18-8 stainless steel stems and 18–8 stainless steel gland and body bolts is required in addition, owing to severe service conditions and corrosive soil action. All valves provided should be required to open either clockwise or counter-clockwise, whichever is the prevalent practice in the municipality.

Hydrants

Fire hydrants should be provided at all strategic locations. Ordinarily the most convenient location is at street intersections, as the hydrant then becomes accessible from four directions. Hydrants are classified as one-way, two-way, three-way, etc., according to the number of hose connections provided. Probably the most widely used is the two-way hydrant with a 3½- or 4-in. pumper connection. The spacing of hydrants varies from about ten per mile of pipe for residential districts to 20 to 30 per mile in heavily congested districts. Allen Hazen suggests that they be spaced on an area basis, that is, the average area per hydrant to be one acre in high-value districts to three acres in residential districts. Freeman suggests that the hydrants should not be located on the plan, but rather on the ground. He suggests further that they be placed not over 500 ft. apart in suburban districts and not over 250 ft. apart in manufacturing districts. Hydrants should be required to meet

A.W.W.A. Standard Specifications for Fire Hydrants for Ordinary Water Works Service—7F.3-1940, and should be painted to show capacity in accordance with A.W.W.A. Specifications for Uniform Marking of Fire Hydrants—7F.3.1-1937. Hydrants should be installed farther back from the curb than has been the practice in the past in order to avoid damage from motor vehicle collisions and should be provided with a separate shut-off gate valve on the connection.

Records

The securing and maintaining of adequate records cannot be too strongly urged. A record of the location of all mains, services, branches, hydrants, valves and all other structures which are buried should be taken in the field and reduced to permanent record form in the office. A committee of the A.W.W.A. studied this matter and recommended a code of standard practice for distribution system records in 1940 (Recommended Practice for Distribution System Records—7G.1-1940). Adherence to such recommendations will insure the municipality of having the information when it is most badly needed.

Appendix

The circle method is very useful where it is required to determine the sizes of the intermediate pipes in a large distribution system. The method requires a knowledge of the economical velocities in the pipes, and there is reproduced in most text books on water supply various formulas for determining the economical velocity. For ordinary sizes of pipe and for average Canadian conditions it can be shown that the economical velocities will run from approximately 3 fps. for 4-in. pipe to 4 fps. for 20-in. pipe where the water is required to be pumped.

The steps in the method are as follows:

1. Taking the grid shown in Fig. 6, assume the demand of the district to be concentrated at a fire at point "A." (In this example take 2,500 gpm. as the fire demand.)

2. Inasmuch as the fire streams from hoses are unsatisfactory when the hose length exceeds 600 ft. in length, draw a circle whose radius is about 80 per cent of this length (say 500 ft.) and whose center is at "A."

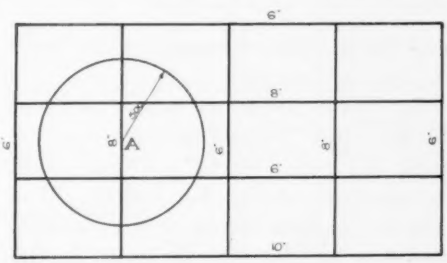


Fig. 6. Typical Portion of a Grid—Example of the Application of the Circle Method of Determining Sizes of Mains

3. Count the number of pipes cut by this circle, in this case six pipes. If the circle is tangent to any pipe or cuts across any pipe, take the number of pipes cut as two for each such instance.

4. Determine the economical velocity. (In this example assume 3 fps.)

5. Determine the diameter of a single pipe that will carry the total demand at the economical velocity. In this example, using 2,500 gpm. at 3 fps., with a coefficient C of 100, the size will be found to be 18 in.

6. In Table 10 there are figures showing equivalent pipe sizes. From this table find how many 4-in. pipes are equivalent in cross-sectional area to

TABLE 10

Equivalent Pipe Sizes

	Size of Pipe in Inches													
	4	6	8	10	12	14	16	18	20	24	30	36	42	48
Number of 4-in. cast-iron pipes having same capacity	1	2.9	6.2	11.0	18	28	39	54	70	113	203	330	497	706
Number of 4-in. cast-iron pipes having same cross-sectional area	1	2.25	4.0	6.3	9.0	12.2	16	20	25	36	48	81	110	144

one 18-in. pipe (twenty 4-in. pipes = one 18-in. pipe).

In this example six pipes were cut by the circle; therefore find from the table six pipes whose combined area is equal to twenty 4-in. pipes. The nearest solution is four 8-in. pipes plus two 6-in. pipes which are equal to $20\frac{1}{2}$ 4-in. pipes.

Six- and 8-in. pipes will, therefore, be installed in the grid so that each circle such as "A" will cut at least two 6-in. and four 8-in. pipes.

Acknowledgments

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Abstracts of Water Works Literature

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HYDRAULICS

Friction Factors for Pipe Flow. LEWIS F. MOODY. Preprint Paper No. 44—SA-4, A.S.M.E. Object of paper to furnish engr. simple means of estg. f in computing h_f in clean pipes with steady flow. f = coef. in Darcy formula $h_f = f \dfrac{L}{D}\dfrac{V^2}{2g}$, in which h_f = loss of head in friction in ft. of fluid; L and D = length and id. of pipe in ft.; V = mean veloc. of flow in fps.; g = acceleration of gravity in fps./sec. (mean value = 32.16). f is dimensionless and function of 2 other dimensionless quants.; $\dfrac{\epsilon}{D}$ = relative roughness (ϵ a linear quant. in ft. representative of abs. roughness), and Reynolds number $\mathbf{R} = \dfrac{VD}{\nu}$ (ν = coef. of kinematic viscosity of fluid in sq. ft./sec.) Fig. 1 gives values of f as function

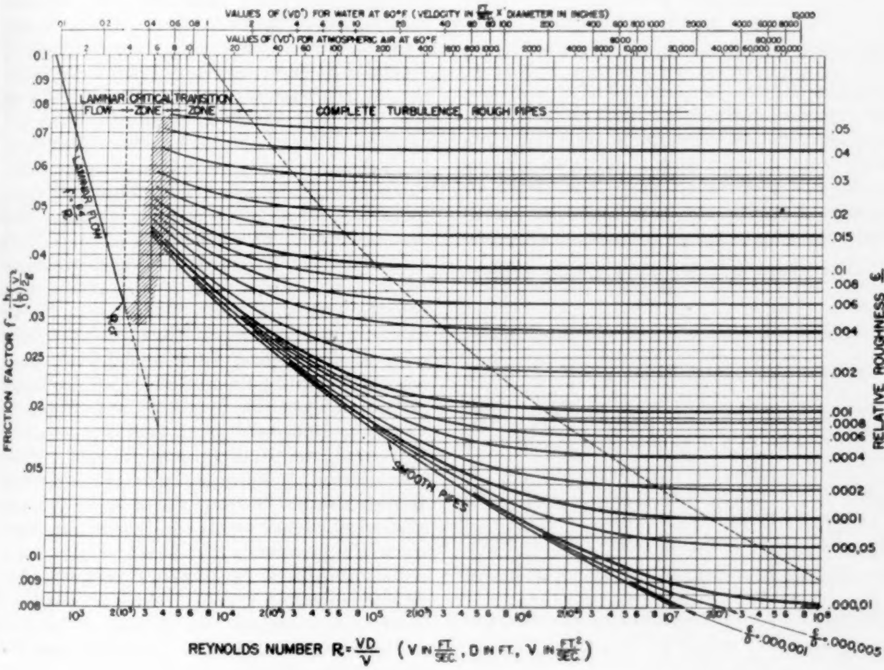


FIG. 1

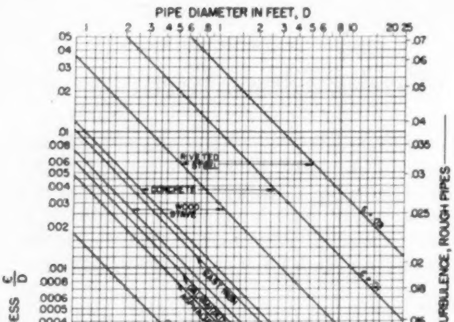


FIG. 2

of **R** and $\frac{\epsilon}{D}$, similar in form to chart of Pigott, published 10 yr. ago and widely used and reproduced in texts, but Fig. 1 closes gap in transition zone of incomplete turbulence between smooth and rough pipe zones. Lines for smooth and rough pipe from work of Kármán, Prandtl and Nikuradse; in transition zone, curves follow Colebrook function $\frac{1}{\sqrt{f}} = -2 \log \left(\frac{\epsilon/D}{3.7} + \frac{2.51}{\mathbf{R}\sqrt{f}} \right)$. Fig. 2 gives $\frac{\epsilon}{D}$ for pipes of various types and diams., also f for complete turbulence where f const. Useful charts not shown here include Fig. 3 giving **R** from D and V; and Fig. 4, giving ν for various fluids and temps. and also **R** from ν and VD. Examples illustrating use of charts given; i.e., find h_f in 200′ 6″ asphalted c-i. pipe carrying water with mean V of 6 fps. For asphalted c-i. pipe and $D = 6''$, Fig. 2 gives $\frac{\epsilon}{D} = 0.0008$. **R** of 2.5 (10⁵) obtained from Fig. 3 (not shown) by computation $\mathbf{R} = \frac{VD}{\nu} = \frac{6 \times 0.5}{0.000012} = 2.5\,(10^5)$ or instead of using **R**, may enter Fig. 1 on top scale with $VD'' = 6 \times 6 = 36$ (V in fps. and D'' in in., gives **R** for water at 60°F.; also for atm. air at 60°F.). For $\frac{\epsilon}{D} = 0.0008$ and $\mathbf{R} = 2.5\,(10^5)$, Fig. 1 shows $f = 0.02$. Then $h_f = f \frac{L}{D} \frac{V^2}{2g} = 0.02 \frac{200}{0.5} \frac{(6)^2}{64.3} = 4.5'$ friction loss.—*Weston Gavett.*

Bed-Load Transportation in Mountain Creek. HANS ALBERT EINSTEIN. U.S. Dept. of Agric. Soil Conservation Service. Investigation made as first step in detg. whether principles established in flume expts. on "bed-load" transportation of sediments applicable in actual streams. Mountain Creek drains 11.7 mi. in Greenville Co., S.C., heads at el. 2047 on Paris Mt. and enters Enoree R. at el. 820. Drainage area comprises 58% woodland; 28% cultivated, mostly cotton; and 14% urban or abandoned. Transported sediments fine with avg. diam. of 0.9 mm. Cross-section of stream is flume-like with flat sandy bottom avg. 14.22′ in width, banks sloping at 45°, 2–3′ high and covered with heavy grass. Slope of channel is 1.11′ in 750′ reach of stream in exptl. area or 0.001476. Duration of expt. was Aug. 19 to Nov. 1, '41, and water depth less than 0.32′ 50% of time and less than 1.0′ 90% of time. Mean annual runoff estd. at 16.45 cfs. *Apparatus:* Avg. distance between points of rest of particles in bed load of stream about 100 times diam. of grain. For Mountain Creek this distance would be about 4″ and probability of distance of 2′ would be 1:500. Hopper with top practically flush with stream bottom, covered with wire mesh, 2′ wide up and down stream and extending entire width of stream, with sloping sides to conc. sediment at point whence it could be readily pumped, indicated as proper size app. to collect bed load, as it would be impossible to collect by sampling. But small depth of sand in stream bottom necessitated 2 hoppers each covering half stream width. Sediment and water pumped alternately from these into measuring tank with straight sides and conical bottom to conc. sediment again. Practically clear water overflowed sides into pipeline which conducted it back into stream. At frequent intervals sediment in bottom of tank flushed back into stream immediately below hopper so that

this portion of stream would have same bed load as that above hopper. Thus, bed would not be eroded by desedimented water and natural slope would be preserved. Tank suspended on spring and as it sank with wt. of sediment its movement recorded. This and other functions of device calibrated. Total rainfall during period 3.55″, of which 1.55″ concd. in one storm. Two short floods produced by releasing water from small mill pond of 4.5-acre-ft. capac. *Hydr. Measurements:* 16 cross-sections at 50′ intervals upstream from hopper and with first 1′ upstream from its upper edge, used to measure slope, position of bed and water surface. Marked permanently by stakes at each end from which wire chain with links 1.5′ long could be suspended across stream. Measurements of el. of water surface and of bottom always made at junction of links. Automatic stage recorders installed at cross-sections 0 and 15. Bed-load formulas and friction laws usually contain slope of energy grade line as one of hydr. variables. Energy grade line found to be practically parallel to water surface. For measuring veloc. local conditions prohibited a weir so done by floats, applying factor 0.85 to get avg. in section. Results checked with pitot tube occasionally and found good. Chem. anal. of sediments made each time collecting tank flushed. *Formulas:* Bed-load formula expresses rate of transportation as function of discharge and of phys. characteristics of bed load; both in terms of unit width of bed. Energy of stream dissipated by friction and turbulence due to roughness of bed and banks. Energy effective in transporting bed load only that transformed into turbulence over bed; that along banks obviously not effective because no bed load to act on. Often assumed that friction along banks can be neglected if width of section more than 10 times depth, but not found so in Mountain Creek where banks relatively very rough. Necessary, therefore, to det. friction along banks separately. Author has devised method for this using Manning's formula and making calcns. separately for bed and banks. This involves 11 variables in various formulas of which 5 measured in taking observations on stream and sixth assumed as later described. This enabled remainder to be calcd. if desired. Coef. of roughness in Manning's formula designated by "n" and total energy dissipation by friction due to wetted perimeter is sum of that due to banks and to bed. Applicable coefs. designated n_w and n_b. n_w may be assumed to be const. and if n_b also const., analytical tests indicated that n_w 0.068 and n_b 0.012. If, however, n_b increases with increased movement of sediment, n_w must be smaller than 0.068 and for that reason various values of n_w later noted used, these being sixth variable. After reviewing various formulas for bed-load transportation heretofore proposed $\psi - \phi$ formula adopted. In this ϕ is intensity of bed-load transportation and ψ is hydr. variable detd. by density of solid sediment and of fluid, representative grain diam. (35% of sample smaller), slope of energy gradient and hydr. radius of bed. ϕ can be interpreted as ratio of veloc. bed load would have if it moved down river as solid layer as thick as representative grain diam., divided by settling veloc. of this particle in water. ψ can be interpreted as wt. of particle acting on portion of bed covered by it. Value of ϕ not dependent on side-wall friction but that of ψ is. Calcd. for $n_w = 0.068$, 0.062, 0.056, 0.050 and 0.044. Expt. has shown that n_b increases with increase in bed load or perhaps that a part of energy of flow used in transporting load while n remains same. Latter joint effect can be evaluated as increase in n_b. Empirical curves which show this, and also curves showing the relationship of ϕ and ψ as derived from flume expts., compared with position of series of platted points obtained from calcns. using exptl. data obtained in Mountain Creek work. Points calcd. for $n_w = 0.056$ coincide with curves very well and that value used in all subsequent calcns. Floods caused by release of water from small reservoir receded much more rapidly than natural floods and points calcd. from data during falling stage of these floods depart widely from curve, due to fact that bottom roughened by flows of higher stages gradually become smoother as stage falls. Adjustment requires time not present after peak of artificial floods. *Application of Results:* Three equations or principles governing flow and transportation on uniform movable bed as derived from flume expts. found applicable to Mountain Creek: (1) Eq. for transportation of bed load; (2) Eq. for roughness of movable bed as represented by $n_b - \phi$ curve; (3) Eq. for distr. of energy spent by flowing water between different parts of wetted perimeter, if parts have different roughness. Thus for flows similar to that of

Mountain Creek capac. for bed load may be calcd. by means of "x" sections of stream and mech. anals. of bed material. Where slope and veloc. measurements not available, they can be calcd. by friction formula, assuming values of n. General principles involved and method used to study effect of changes in different characteristics for reach from which exptl. data secured. Idealized "x" section of same dimensions as natural used. Found that : (1) carrying capac. higher for narrow bed at low stages and for wide bed at high stages. Normal for low stages. Abnormality for high stages explainable from roughness of banks, which, as stage increases, absorb increasing part of energy; (2) transporting capac. would be increased 2.5 times if n_w decreased from 0.068 to 0.044 while stage also would be decreased some. Thus, if bank vegetation decreased, deposition on bed would be decreased, and if addnl. shrubbery planted on banks, scour in bed would be decreased. Roughness of bank very important factor in transportation of bed load; (3) most efficient slope of banks between vertical and 1:1, but in most cases this is too steep to be stable. Change in bank slope from 1:2 to 1:1 doubles transporting capac.; (4) 20% change of slope (uniform flow) will about double transporting capac. and vice versa. If bank slopes changed from 1:2 to 1:1, slope of stream would have to decrease about 20% if transporting capac. remains same. If n_w changed from 0.044 to 0.068, slope would have to increase about 27% if transporting capac. remains same. Basic but unproved assumption underlying procedures described is that stream always carrying capac. bed load. If supply of sediment insufficient or if particular grain sizes lacking, effective grain diam. may change and problem becomes hopeless. *Amount of Average Annual Transportation:* Duration curve as used in planning hydro-elec. power plants good tool in calcg. this. Found that such curves expressed in percentage of mean fairly identical for same region for higher flows. As banks of Mountain Creek 2′ in height (min.) at higher stages, water floods out of stream and that portion of water section has little transporting ability. Annual bed transportation of stream in this reach under natural conditions is 71,000 cu.ft. of sediment. If banks raised 6″, sediment transported would be increased 15% and at same time flooding would be decreased. Same increase in transportation could be secured by decreasing n_w only slightly from 0.056 to about 0.052. Little gain in having bank slope steeper than 1:1. Increase in slope of stream about 5% would increase carrying capac. about same as raising height of banks 6″. *Practical Applications:* (1) Detn. of annual load in different tributaries to locate most important source; (2) effect of changes in watershed on deposition of sediment and effectiveness of measures to counteract effect of changes; (3) final equilibrium of stream bed profile which is in process of change; and (4) necessary "x" section of channel formed in cutting off bend of stream. *Miscellaneous:* As evaluation of measurements in Mountain Creek based on choice of representative grain diam. of bed mixture selected by primitive empirical rule, choice checked by flume measurements using original Mountain Creek sand. Points thus secured platted on normal ϕ/ψ graph indicated that diam. chosen correct. West Goose Creek is one of streams of Tallahatchie R. Basin in Miss. Its "x" section very similar to Mountain Creek but avg. diam. of bed material 0.33 mm., slope (reach in which expts. made) averaging about 0.003, veloc. at lowest stage during expts. 1.65 fps. as compared with 0.9 mm., 0.00147 and 1.25, respectively for Mountain Creek. As these characteristics indicated values of ϕ several times those of Mountain Creek, similar investigation made in Mar. '42. Measurements extended range of ϕ from previous value of 1 up to 10. Believed to be permissible assumption that this extension can be used for sediment coarser than West Goose Creek, thus enabling calcn. of bed-load transportation for rivers several ft. deep and with sands approx. 1 mm. in diam. Validity of assumption must be proved by direct measurement.—*Harold Conkling.*

Surge Control in Pipelines. Charles F. Lapworth. Trans. Inst. Wtr. Engrs. (Br.) **59** ('44). Extended use of automatic control of electrically-driven pumping plant has shown that pressures generated in stopping and starting can far exceed normal pumping pressures and sometimes reach excessive figures. Similar conditions occur with involuntary shutdown following failure of current. Author has carried out tests on several pumping installations. Paper describes results and gives methods of calcg. approx. values of surge pressures to be expected. Deals with various methods of surge control and describes tests on installations to

which these methods applied. Conditions which may give rise to excessive pressures: (1) Stopping of pump. (2) Failure of elec. power. (3) Starting of pump. (4) Sudden opening or closing of sluice valve in pipeline. (5) Slamming of check valve. (6) Cavitation or formation of partial vacuum at high point in pipeline. Subsequent reunion of water column may cause excessive pressures. Veloc. of pressure wave in large reservoir about 4700 fps. Veloc. of pressure wave (in closed pipe) is:

$$a = 12 \div \sqrt{\frac{w}{g}\left(\frac{L}{k} + \frac{d}{Ee}\right)}$$

in which d = diam. of pipe in in.; w = wt. of water in lb./cu.ft.; g = gravity; e = thickness of pipe wall in in.; k = modulus of elasticity of water, 300,000 psi.; E = modulus of elasticity of pipe wall, for steel 29.4×10^6; cast iron 15×10^6 psi.; L = length of pipe in ft. In suction mains, if speed of pump reduced to V_1, pressure rise is $h = \frac{a}{g}(V_0 - V_1)$ in which V_0 = original veloc. of water in pipe and a = veloc. of pressure wave. True so long as speed reduction completed in less time than reflection period of pipeline. If stop takes place in longer time than $2L \div a$ pressure rise will not be so great. Evaluation of max. pressure rise under these conditions best treated graphically by means of diagram in terms of veloc., head and time, known as *VHT* diagram. In general, effect of pipe friction is to introduce damping effect so that max. pressures less than otherwise. All pipelines having same const. K behave in similar manner. $K = aV_0 \div 2gH_0$, where H_0 = initial head. When rate of retardation of water column uniform on valve closure, other variables can be expressed by $N = aT \div 2L$ where T = time of closure of valve. Pressure rise for varying values of K and N shown. Ordinary sluice valve does not give uniform retardation if closed uniformly. Ideal check valve should seat at exact moment water column at rest, before reversal occurs. Acceleration of reversal of rigid column of water is $H_0 g \div L$. Some cases in which no check valve can close sufficiently quickly. Where water column near pump in delivery main comes to rest in time less than reflection period in pipeline, drop in pressure is $aV_0 \div g$. Ignoring pipe friction, subsequent rise in pressure is same. Vacuum pressures cannot occur unless pipeline const. K greater than $\frac{1}{2}$. Summary of results of surge tests in delivery mains at Stevenage and Macclesfield shows that: (1) If design of check valve such as to elim. slamming *VHT* diagram gives approx. values of max. pressures. (2) Slamming of check valve can raise max. pressure considerably. (3) Where line of min. pressure lies above profile of pipeline, *VHT* diagram gives approx. min. pressures at pumping station. (4) Where line of min. pressures lies below profile of pipeline, min. pressures greater than those given by VHT diagram. In general, problem of surge control to increase or decrease veloc. of water column more slowly or to dissipate energy of moving column harmlessly. Methods proposed: (1) increasing inertia of moving parts of pump and prime mover by flywheel; (2) motorized valve which simulates hand operation by an attendant; (3) electrically-operated, delayed-action stop/start on motor; (4) pressure-relief valve; (5) surge supressor; (6) air vessel. Where cavitation occurs at high points on pipeline: (7) surge tower; (8) surge tank; (9) surge buffer. If flywheel added so that after period of $2L \div a$ after shutdown pump revolving at $\frac{3}{4}$ speed, max. pressure generated reduced. Flywheel not applicable to large installations. For normal purposes, where veloc. in pipeline 2 to $2\frac{1}{2}$ fps., normally sufficient to limit time of closure of sluice valve to 15 to 30 times reflection period of pipeline. Equal to about 37 to 75 sec. per mi. of pipeline. Delayed action stop/start more readily adaptable to pumping plant having slip-ring motor; gives no protection against current failure. Pressure relief valve fixed to tee-branch on pipeline, size being normally $\frac{1}{3}$ to $\frac{1}{2}$ diam. of main pipe. Surge suppressor is relief valve in which port arranged to open before rise in pressure occurs. Period of subnormal pressure used to open valve. Suitable only where subnormal pressures occur before rise in pressure. Air vessel is steel cylinder connected to branch of main. Effect is to damp surge effect by elasticity of air in vessel. Period of mass oscillation (T) of water column is $2\pi\sqrt{wLC_0 \div gAP_0}$ where C_0 = vol. of air under static head, cu.ft.; P_0 = pressure of air under static head, lb./sq.ft. absolute; w = wt. of water in lb./cu.ft.; L = length of main in ft.; and A = area of main in sq.ft. Calcns. of size of surge tower required made by assuming cross-sectional area and detg. rise and fall of water level in tower. Water column con-

trolled by surge tower oscillates as mass rather than elastic body. Inertia head produced by acceleration or retardation of column of water leading to tower is (length of pipe in ft.) × (acceleration in pipe, fps.) ÷ g (fps./sec.). Surge tank similar to surge tower except that its use not confined to points near hydr. grade line. Use of surge buffer, like surge tower, confined to places in pipeline where cavitation would occur. *Discussion.* Wtr. & Wtr. Eng. (Br.) **47**: 387 (Sept. '44). J. S. Blair: Writer has found convenient method for taking high-speed records on circular chart app. This consists of small synchronous motor with shaft running at 1 rpm., shaft ending in circular rubber disc. Required speed of rotation of chart can be obtained by sliding motor to or from center of chart. Slowing of pressure wave at Macclesfield tests may have been due to dissolved air released by low pressure. Some simple type of mechanism to prevent pumps from starting in such succession that peak surges superimposed would seem to meet case. Introduction of damping by throttling outlet of air vessel has marked effect in reducing amplitude of surge and causing it to die out more rapidly than normally. Adjustment best found by trial and error, since too much friction would result in air vessel being relatively inoperative and cavitation occurring in main. Horizontal air vessel seems undesirable, since there is considerable wave action inside horizontal vessel resulting in possibility of inlet being uncovered. Vessel also requires more floor space. R. W. S. Thompson: In large pipes of avg. thickness pressure wave travels at roughly ¾ mi./sec. For every fps. of veloc. suppressed during reflection period, shock pressure of 50 psi. caused. In relatively short pipeline or where diam. of valve less than main, effective time of closing may be larger percentage of actual time than was case in instance cited by author. Two causes of surges, other than mentioned by author are refilling of empty mains and automatic self-closing valves. To limit shock to 25 psi. when long main being used to fill short length, veloc. in pipe supplying water must not exceed 0.5 fps. when filling is complete and last air valve closes. When long length is being filled from shorter length, max. surge pressure will be less than that corresponding to veloc. of supply pipe at moment when filling completed. Where pipe being filled from open end there is no shock. *Author's reply:* Blair correct in pointing out that relationship between quant. discharged and percentage opening of sluice valve refers to one valve on particular length of main and cannot be taken as representative of other mains. General shape of curve will be similar. Smaller air vessels such as are fitted to reciprocating pumps normally vertical; larger type normally horizontal. Where practicable, vertical type preferred. *Discussion. Ibid.* **47**: 448 (Oct. '44). F. A. Klouman: Subject to certain variations dependent on constr. of valve, loss of head across gate valve may be determined from:

$$H_f = \frac{V^2}{2g}\left(\frac{A}{aK_d} - 1\right)^2$$

where H_f = loss of head, V = veloc. of approach, A = area through valve when wide open, a = area of waterway past partially closed door, and K_d = discharge coef. This formula shows that with valve same size as pipe in which it is fitted, and with low pipe velocs. customary in water works practice, amt. of throttling necessary to produce appreciable loss of head considerable. Explanation of small effect produced by sluice valve in early stages of closing. In varying degree same true of any other type of valve. Position of valve at which real throttling takes place, giving rise to spouting veloc. through valve, may be approx. detd. from:

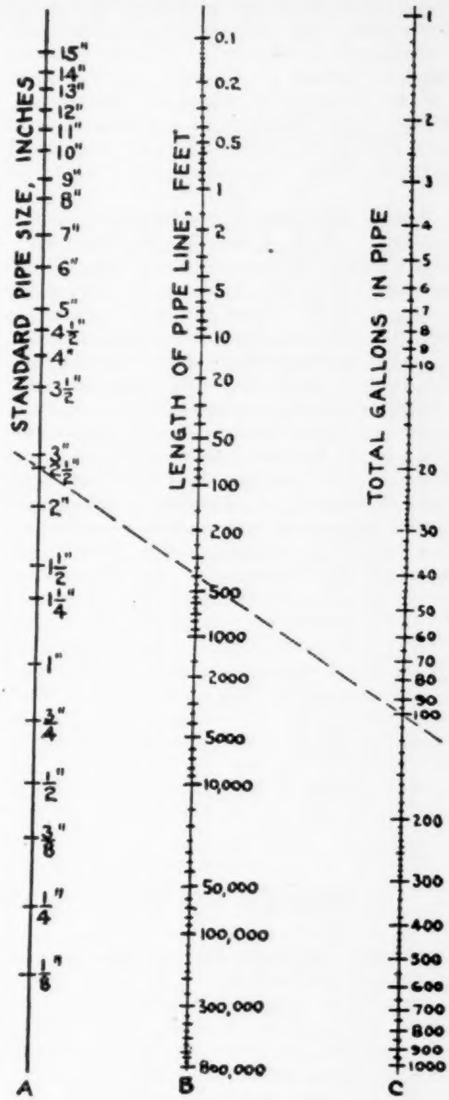
$$a = \frac{AV}{K_d\sqrt{2g(H-h)}}$$

where a = area of waterway at commencement of throttling, A = cross-sectional area of pipe, V = veloc. in pipe at commencement of throttling, H = static head, h = total pipe friction at veloc. V, and K_d = discharge coef. of valve. In selecting valve for conditions which require surge to be taken into acct., smallest possible valve should be chosen consistent with its satisfying other important criteria, such as head loss. If area of valve opening at start of throttling more than 30% of full area, ordinary sluice valve and streamlined plunger valve have best characteristics. If throttling area less than 30%, spectacle eye or butterfly types preferred. Motorized valve gives no protection against involuntary shut-down following current failure. This can be overcome by substituting hydr. cylinder in place of motor, pressure for operating piston being taken from main. J. H. Dossett: Writer has recently made pressure records showing effect of surge control on suction main at Slaughter Hill Booster Sta-

tion of Nantwich Rural Dist. Council. Suction main consists of 4200′ of 7″ and 4200′ of 9″ main. Booster plant automatically operated and originally equipped with two-step starters controlled by pressure switch. Time of start approx. 5 sec. When pumps cut out main pressure rose from 67 to 170′ and on restarting pressure reduced to zero. Surge conditions transmitted back to Eaton borehole pumping station and upset working of chlorination plant there. Two-step starters replaced by 12-step starters requiring 60 sec. for both starting and stopping. Stopping caused rise from about 110 to 140′, and drop from 100′ to about 60′ on restarting. A. B. Baldwin: Diam. of pipe affects consideration of water hammer only insofar as repercussions of burst large main generally more far reaching than in case of small main. Inclination to decide time of closure according to diam. of pipe rather than on length of pipe to "free end" at variance with underlying principles. Noted that rise in pressure bears no relation to static head on main. "Concussion pipe," adaptation of air vessel for remedial measures on domestic installations, may be of interest. It is simply 2′ length of vertical pipe fitted to rising main at point of feed to ball valve of gravity tank. Water level set some 18″ below top of pipe and open end plugged off. Air space maintd. by resetting water level annually.—*H. E. Babbitt.*

Chlorine—A Critical Material Solves a Critical Problem. L. A. Jackson & W. A. Mayhan. W.W. & Sew. **90**: 4: 13 (Apr. '43). Little Rock, Ark., filter plant supplied from Lake Winona through 33-mi., cement-lined, 39″ id., steel-cylinder, reinforced concrete pipeline. Original $C = 147$ at 25.32 mgd. Year later $C = 120$ at 20.38 mgd. Inspection showed mat of slime consisting of encapsulated bacteria and *Crenothrix*. Chloramination only slightly improved capac. No improvement with $CuSO_4$. Continuous break-point chlorination and occasional very heavy dosages of chlorine slowly improved capac. until avg. C of 137 reached. Other advantages: tastes and odors non-existent and sterile water throughout plant.—*F. J. Maier.*

Handy Table Gives Pipe Volumes. W. F. Schaphorst. The Driller **7**: 6: 21 (June '43). Often desirable to know vol. of given pipe length in gal. Accompanying chart provides for all std. pipe sizes from ⅛ to 15″ in Column



(Col.) A, and any length from 0.1′ to 800,000′ in Col. B. Col. C shows vols. from 1 to 1000 gal. To use chart: run straight line through pipe size, Col. A, and length of pipeline, Col. B. Intersection with Col. C immediately gives total no. of gal. in pipe or pipeline. Conversely, length of pipe required to hold given no. of gal., or pipe size necessary

to hold given no. of gal., within certain length limitations, may be detd. Example of "trick" use: To det. vol. of ½″ pipe 1′ long, run line through ½″, Col. A, and 10,000 in Col. B. Intersection with C at 160 gal. for 10,000′ line. Pointing off 4 places to left gives 0.016 gal. as vol. of ½″ pipe 1′ long.—*Ralph E. Noble.*

Water Hammer Correctives. Richard Bennett. W.W. & Sew. **91**: 69 (June '44). Water hammer in pipeline due to h-p. waves developed when flow suddenly checked, as in closing a valve. Water hammer troubles expected to increase due to customer demand for quick-closing faucets and fixtures. Relief valves, which generally operated only under high surge pressures, have been used at pumping stations and hydro-elec. plants to elim. water hammer where water spill no problem. Air chambers also used, but difficulty of keeping air in chamber makes this device impractical unless given close attention, or air recharging can be done automatically with compressors. Relief may also be obtained by use of surge suppressors which operate similarly to automobile shock absorbers cushioning impact of water rebounding in piping system. In principle, yielding surface exposed to h-p. wave which is thus reflected back in diminished intensity until finally dampened. In Wacor Water Hammer Arrester this yielding surface in form of closed collapsible metal billows filled with special compressible emulsion. Work done on bellows responsible for dampening of hammer pressure. In Josam Shock Absorber yielding surface provided by diaphragm moving against compression of 4 coil springs. Suppressors should be placed near valve causing water hammer and with openings at bottom to avoid hazards from sediment being washed into casing. Water hammer is problem in centrifugal pump installations where use of flap type check valves increases effect of surge waves and water hammer. Slow-closing check valves impractical, even though theoretically correct, and even if there is little or no audible water hammer, there may be pressure rises dangerous to pipelines and fittings. Check valves should close before pressure reverses from immediate low to quickly ensuing high, and quick-closing valves have been found, in many cases, to give proper service. Smolensky check valve, Rensselaer's power valve, Chapman's tilting disc, and Larner-Johnson examples of quick-closing automatic check valves. In many cases necessary to install air tank or relief valve in addn. to check valve. Pelton Surge Suppressor, essentially relief valve, found efficient in combination with Larner-Johnson check valve in Boulder Dam water supply to Boulder City. For studying and recording surge conditions in piping systems, author cites use of Bachrach Chronomatic Drum and Portable Recorder which give pressure-time diagram and have unrestricted speed selectivity and instantaneous speed indication.—*P.H.E.A.*

CHEMICAL FEEDING, CONDITIONING AND SEDIMENTATION

Judging Coagulants for the Water Supply of the Army in the Field. V. A. Yakovenko. Voenno-Sanit. Delo (U.S.S.R.) **8**: 100 ('40); Chem. Zentr. (Ger.) **I**: 2297 ('41). Cl exerts good oxidizing action on org. constituents of water, especially on those of animal origin. Latter effect also observed to slight deg. when $Fe_2(SO_4)_3$ used, effect being directly proportional to amt. of coagulant. Upon soln. of ppt. with H_2SO_4 the adsorbed org. material again goes into soln. but oxidizability of water then less than before pptn., especially if much org. material of animal origin present. Fe salts useful in practice as coagulants since they also remove Mn and As, not sensitive to temp. and form cryst. $Fe(OH)_3$ of density 3.6 at practically any pH. $Al(OH)_3$ amorphous and has lower density (2.4) so that pptn. slower and occurs only at pH 5.4–8.5 (at pH 4–5, $5Al_2O_3 \cdot 3SO_3$ forms, predominately effective for removing color), with effect being principally a reduction of turbidity. Addn. of Na_2SiO_3 to improve coagulating effect of Al salts makes filtration more difficult.—*C.A.*

Silica Sol as Auxiliary Coagulant With Copperas. A. A. Hirsch. Ind. Eng. Chem. **35**: 811 (July '43). Graphic anal. of lab. jar test results on Lower Mississippi R. water sought to define optimum economic proportions of silica sol and copperas, used as coagulant pair, for any desired turbidity level in

settled water. Silica sol prepd. by original Baylis method. Added alone in raw water, silica sol acted as dispersing agent. In conjunction with partial lime softening, silica sol produced clearer settled water when fed preceding lime than when applied after lime. When silica sol preceded copperas in raw water, floc break and coagulation materially enhanced, except at low dosages of copperas. After prelim. lime softening, activated silica superior to copperas for production of clear settled water; however composite coagulation surpasses results with either single chem. Silicated copperas floc initially more voluminous than plain copperas floc, but shrank to more compact and tough mass on standing. Paper filtration of supernatants gave clearer effluents from bi-coagulant treatment. Data plotted with coagulant dosages as co-ordinates and residual turbidity as parameter; line through point on each axis located proportional to dollar per ton cost of other chem. established so-called "equal-cost slope," whose loci of tangents to turbidity curves marked most economical proportions to attain desired final turbidity. Relatively high indicated amt. of activated silica, economical in lime-softened water, virtually placed it in role of main coagulant, with copperas functioning more as auxiliary. Improved plant performance at Algiers, La., following introduction of activated silica preceding copperas dose, noted.—*A. A. Hirsch.*

Two-Stage Treatment Provided at the Marceline Water Purification Plant. GLENN C. FOX. J. Missouri W. Sew. Conf. **15:** 3: 21 ('44). In original plant layout, 2 basins operated in parallel and only one dosage point provided. Remodeled plant provides flocculation and sedimentation in each of 2 settling basins with provision for either series or parallel operation. Reduction of approx. 50% in cost of chems., as well as 50% reduction in total solids in finished water. Reduced load on filters permits longer runs with considerable saving in wash water.—*C.A.*

The Filtration of an Acid Water, Using Sodium Aluminate as a Coagulant. TOM WATERTON. Wtr. & Wtr. Eng. (Br.) **47:** 446 (Oct. '44). From Inst. of Water Engrs. Paper describes successful expts. carried out at Thrum Hall filtration plant of Halifax Corp. Waterworks. Crude water being acid, seemed logical to assume it could be utilized to ppt. aluminum hydroxide from alk. sodium aluminate, whereas artificial alkalinity had to be induced to obtain aluminum hydroxide from aluminum sulfate. Astonishing results obtained in lab. using sodium aluminate alone. Complete clarification achieved using dose of 0.05 gpg. (Imp.) of sodium aluminate and nothing else. Saving in chems. alone on plant delivering 7 mgd. (Imp.) might be £600 to £700 annually. Since working conditions can never be accurately imitated in lab., some expense justified in exptl. plant for verification of results. Two 9′ diam. pressure filtration units segregated from rest of plant. Small sodium aluminate plant installed and lab. expts. repeated with immediate success. High flows, low flows, sudden increases and decreases in flow all tried to test stability of floc. None succeeded in destroying eff. of filtration. Results of treatment on all 84 filters same as obtained on exptl. plant. Following results obtained:

	Old Treatment		*New Treatment*
	Aluminum Sulfate	*Slaked Lime*	*Sodium Aluminate*
Chem. dose, *gpg.*	0.7	0.25	0.05
Cost per ton, £	8.5.3	2.8.0	37.10.0
Cost per mil.gal. (Imp.)	7s 4½d	9¼d	2s 4¾d

Wash water greatly reduced and phenomenally low (varying from 0.50 to 0.88%). Found that if pH of treated water fell below 5.6, free alumina likely to appear in filtrate. Adjustment of pH could be effected by dose of ⅓ gpg. (Imp.) of lime. Points of interest worthy of note: (1) Sodium aluminate more readily sol. than aluminum sulfate; (2) sodium aluminate packed in water-tight drums and is cleaner to handle than sulfate blocks; (3) sodium aluminate solns. cleaner than commercial aluminum sulfate solns. and pipelines do not choke with former; (4) filter beds appear much cleaner than formerly and tendency to form peat "buttons" on surface has disappeared; (5) indications are that corrosion of tanks, fittings, pumps, etc., considerably less.—*H. E. Babbitt.*

Corner-Cutting Practices

at the St. Louis County Water Co.

By W. Victor Weir

Supt., St. Louis County Water Co., University City, Mo.

Small Meters Repaired "En Masse" Instead of Singly

When $\frac{5}{8}$- and $\frac{3}{4}$-in. meters are brought to the shop for overhauling the following procedure is followed if the meters have interchangeable parts:

(1) After testing, the meters are torn down and various units are placed in proper containers, all registers together, all trains together, etc.
(2) A repairman overhauls one type of unit at a time, i.e., the trains in one container are repaired, then the chambers in another container are properly fitted with discs, etc.
(3) After cases have been properly cleaned, the meters are assembled at the assembly bench where all necessary properly repaired units are available within arm's reach.
(4) Meters are then tested at proper rates of flow. Only the occasional meters which do not pass accuracy tests are treated individually.

This procedure has enabled repairmen practically to double their output over that experienced when meters were overhauled individually. The fact that parts seldom get back into the case from which they were removed does not complicate meter repairing, provided the parts are truly interchangeable.

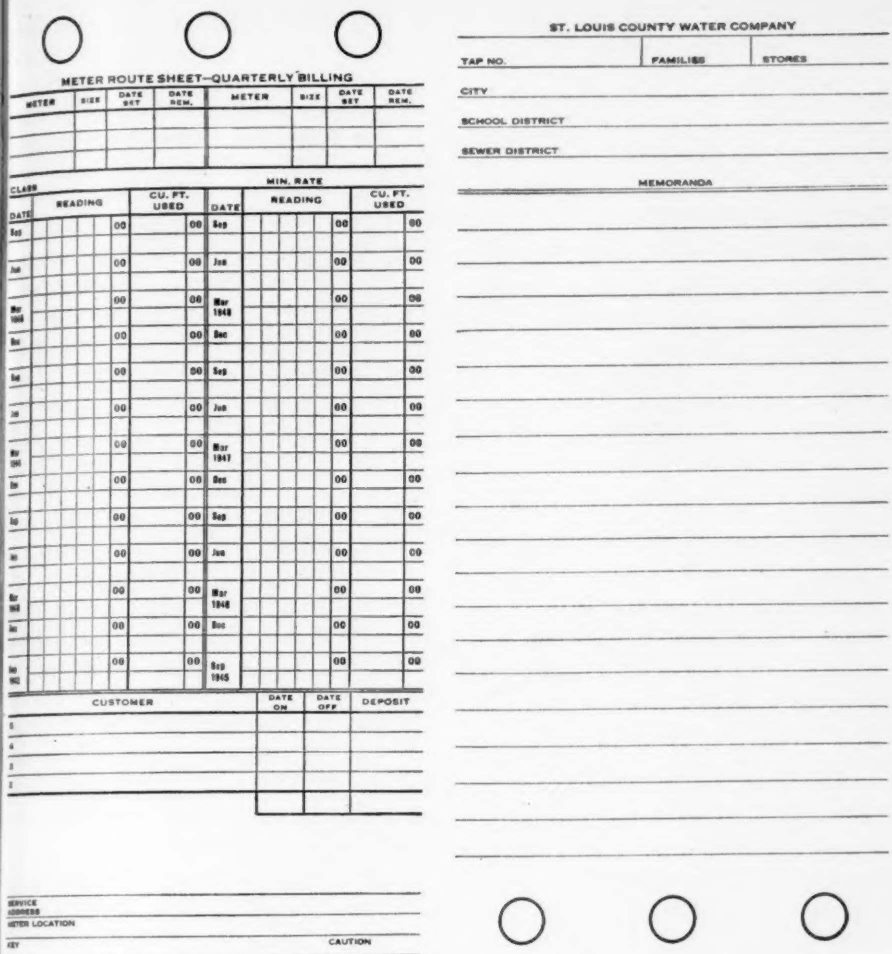
Small Meter Parts Cleaned by Tumbling

Small metal parts are cleaned of scale, rust and dirt by tumbling with steel shot in a trisodium phosphate solution in a midget (Sears-Roebuck) concrete mixer. A $\frac{1}{3}$-hp. motor with a V-belt sheave, belted to the flat pulley on the mixer furnishes the power. A minimum of manpower produces excellent cleaning results.

Redesigned Meter Book Sheets Last Twice as Long

Sheets $4\frac{1}{4} \times 9$ in. previously contained spaces for thirteen readings. They were redesigned to cover 26 readings, with an equal number of blank lines to allow entering intermediate readings, etc. Doubling the capacity of each sheet cut the labor of preparing new sheets in half. A longer and therefore more valuable record is also shown on each sheet.

Space is also provided for showing the names of new customers when occupancy or ownership of premises changes. It is thus unnecessary to



Redesigned Meter Book Sheets in Use at the St. Louis County Water Co.

The spaces provided have been doubled from thirteen to 26. Blank lines are allowed for intermediate readings. Spaces are provided for the names of new owners.

make up a new meter book sheet each time a service is billed in a new name.

Piece Rates Instead of Hourly Rates for Laying Pipe

Workmen will produce much more if they have the proper pay incentive. The establishment of proper piece rates for digging and backfilling trenches and for installing pipes will increase man-hour output from 50 to 100 per cent. Our piece rates have been set to enable men to earn from 25 to 50 per cent more than their regular hourly rate of pay, and our cost per foot is materially lower than if work were done at an hourly rate. Proper piece

rate wages will enable workmen to earn wages comparable to war plant wages and will cause increased output without requiring more men, all without increasing the unit cost of installing mains.

Mechanical Joint Sleeves for Water Main Repairs

Rubber-packed, mechanical-joint solid and split sleeves are used instead of poured-joint sleeves in repairing broken mains. Although the sleeve cost is higher, the time saved in making a repair and the saving in manpower cost more than make up the difference.

Complete Sets of Valve Measurements for All Foremen

Prior to the preparation of a valve record system, similar to the "plat and list" type of records recommended by the A.W.W.A., it was necessary for foremen to spend a considerable amount of time obtaining valve measurements from the office plat books. Each foreman now carries a book containing all valve records for the 885 mi. of mains, entirely eliminating trips or phone calls to the office for valve information.

Use of Portland Cement in Backfilling With Wet Earth

Excavations made in repairing leaks often must be backfilled with the muddy earth removed from the hole and guarded for several days until the excess water soaks into the earth, or backfilled with dry earth obtained elsewhere, with the subsequent hauling away of the wet earth removed from the hole.

To stabilize quickly wet-earth backfill, Portland cement is mixed with the earth as it is shoveled into the hole. The cement is sprinkled on the wet earth and pugged to mix it intimately and to absorb the excess water. One or two sacks of cement properly mixed with the backfill in an ordinary leak excavation will cause the backfill to be solid enough to support traffic as soon as the cement has set. No lanterns or guards need be left on the excavation, and it is consequently unnecessary for men to return to the excavation for several successive days, lighting and replacing lanterns or guards.

The cost of cement is much less than the cost of manpower to obtain dry dirt or to protect the excavation from traffic for several days.

The foregoing summary of new methods, introduced at the St. Louis County Water Co. in order to save war-scarce manpower and materials, was submitted to Association headquarters as a result of the Secretary's communication of February 21, which went to the entire membership under the heading of "Wartime Water Works Information."

Corner-Cutting Practices
at the Brooklyn Union Gas Co.

Derived from The Brooklyn Union Gas News and a communication from Thomas J. Perry, Brooklyn Union Gas Co., Brooklyn, N. Y.

THE Brooklyn Union Gas Co., Brooklyn, N.Y., is the parent of the recharger for dry-cell flashlight batteries which was described in the Wartime Water Works Information communication from the A.W.W.A. Secretary to all members, under date of February 21. The company reports a saving of 67 per cent.

When Thomas J. Perry, Superintendent of Customer Service, noted in the July 29, 1943, issue of *Gas Age* an article which told of a method of recharging dry cells developed by the Pasadena Police and used with success by the Southern California Gas Co., he assigned Edward C. Hollowell, Superintendent's Assistant, to investigate.

Since the process was to be simply a matter of reactivating the cells by passing a direct current through them at a slow rate over a period of hours, Hollowell began experimenting with materials that were easily available. First used was a small motor generator set and a milliampere meter that were readily at hand. Fifteen discarded cells whose casings showed no signs of corrosion and that measured 2 amp. or better on a short test were subjected to a six-hour charge at the recommended rate. Preliminary examination of the recharged cells indicated that they were as efficient as new cells, but as a further test four were forwarded to the chemical laboratory for the test usually applied to new cells being considered for purchase. Two cells were also forwarded from new stock as pilots for comparison. Results of the laboratory test indicated that the four cells that had been recharged were equal in all respects to the new cells.

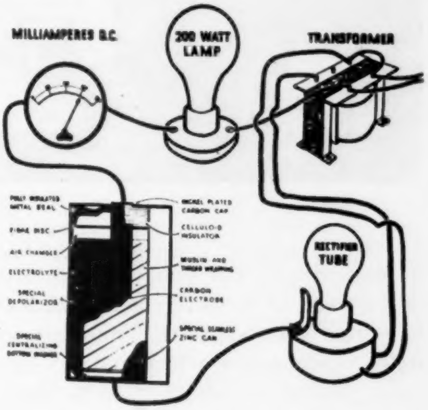


FIGURE 1

On the strength of these tests and of experimental field use of recharged cells, apparatus consisting of a cabinet of drawers in which cells are placed for recharging was built. The device is composed of a rectifier tube, which was previously used in a sprinkler system, to convert alternating to direct current; a complimentary transformer,

FIGURE 2

in lieu of a motor generator, to step down the 110 v. to 75; a 200-w. bulb in series for resistance; and an old milliampere meter which was to have been junked as of insufficient range. During the last half hour, a second 200-w. bulb is connected to the circuit to compensate for the increased resistance offered by the dry cells (Fig. 1).

The cost of constructing the cabinet (Fig. 2) was $40. It contains four drawers, each accommodating 50 cells. Each drawer is in parallel and the cells in each drawer are in series. There is a bus bar in the front and in the back of each drawer, each adjusted to hold the cells tightly in place and to act as a current distributor. The milliampere meter that serves as a guide to the charging rate applies only to the top drawer. The actual power used to charge 200 cells for six hours is 945 whr. This adds less than one cent to the electrical energy costs each time the apparatus is used.